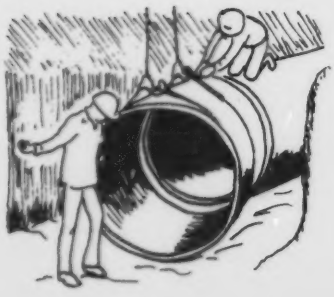
STORMWATER MANAGEMENT GUIDELINES

for the

Province of Alberta



Alberta
ENVIRONMENT

87-03380/

STORMWATER MANAGEMENT GUIDELINES

FOR THE

PROVINCE OF ALBERTA

Prepared By:

I.D. ENGINEERING COMPANY LTD.
MARSHALL MACKLIN MONAGHAN LTD.
STANLEY ASSOCIATES ENGINEERING LTD.

Prepared For:

ALBERTA ENVIRONMENT
STANDARDS AND APPROVALS DIVISION
MUNICIPAL ENGINEERING BRANCH
4TH FLOOR, 9820 - 106 STREET
EDMONTON, ALBERTA
T5K 2J6

MARCH 1987

ACKNOWLEDGEMENTS

This publication was prepared for the Standards and Approvals Division of Alberta Environment under contract to a team of consulting engineers including John Hodgson from Stanley Associates Engineering Ltd., Russ Lewis from Marshall Macklin Monaghan Ltd. and Clive Bright from I.D. Engineering Ltd. The development of the publication was coordinated by a steering committee consisting of Maq Ahmad from the City of Edmonton, Steve Lee from the County of Strathcona and staff from Alberta Environment. Major Alberta Environment reviewers were Pat Lang and Lawrence Hermanutz.

Note: The mention of trade names or commercial products in this publication is for illustration purposes only and does not constitute either an endorsement or a recommendation by Alberta Environment.

PREFACE

This publication is intended to help municipalities, local authorities, consulting engineers and developers in the planning and design of stormwater management systems in Alberta. It outlines the objectives of stormwater management and the available methodologies and the concepts for the planning, design and operation of drainage systems. The publication also describes the regulatory framework which applies to drainage system design and construction.

TABLE OF CONTENTS

		Page
ACKNOWLEDGEMENTS		(i)
PREFACE		(ii)
TABLE OF CONTENTS		(iii)
LIST OF TABLES		(vii)
LIST OF FIGURES		(viii)
1.0	INTRODUCTION	1.1
2.0	STORMWATER MANAGEMENT IN PERSPECTIVE	2.1
2.1	THE EVOLUTION OF STORMWATER MANAGEMENT	2.1
2.2	THE ONTARIO EXPERIENCE	2.3
2.3	THE WINNIPEG EXPERIENCE	2.6
2.4	THE BRITISH COLUMBIA EXPERIENCE	2.10
2.5	THE ALBERTA EXPERIENCE	2.13
3.0	APPROVAL CONSIDERATIONS	3.1
3.1	ROLE OF ALBERTA ENVIRONMENT	3.1
3.2	OBJECTIVES OF ALBERTA ENVIRONMENT	3.1
3.3	SUBMISSION REQUIREMENTS	3.2
3.3.1	Clean Water Act	3.2
3.3.2	Water Resources Act	3.3
3.3.3	Approval of the Local Authority	3.5
3.3.4	Approval Coordination	3.6
4.0	PLANNING FOR STORMWATER MANAGEMENT	4.1
4.1	CONSIDERATIONS FOR RURAL DRAINAGE SYSTEMS	4.1
4.1.1	Road Drainage	4.2
4.1.2	Agricultural Drainage	4.2
4.1.3	Urbanization in Rural Watersheds	4.4

		Page
4.2	DRAINAGE CONSIDERATIONS FOR URBAN AREAS	4.5
4.2.1	The Impact of Development	4.5
4.2.2	Convenience Drainage For Urban Areas	4.5
4.2.3	Flood Protection in Urban Areas	4.6
4.2.4	Storage For Flood Protection	4.6
4.3	DRAINAGE PLANNING IN CONTEXT OF A WATERSHED	4.7
4.3.1	General	4.7
4.3.2	Watershed Drainage Plans	4.12
4.3.3	Master Drainage Plans	4.17
4.4	THE DEVELOPMENT OF DESIGN STANDARDS	4.20
5.0	**DRAINAGE SYSTEMS FOR URBAN AREAS**	
5.1	THE DUAL DRAINAGE CONCEPT	5.1
5.1.1	The Minor System	5.1
5.1.2	The Major System	5.3
5.2	LEVEL OF SERVICE	5.3
5.2.1	System Behaviour During Minor Storms	5.4
5.2.2	System Behaviour During Major Storms	5.5
5.2.3	System Behaviour Above Capacity	5.5
5.3	DESIGN CAPACITIES	5.6
5.3.1	Minor System Design Capacity	5.6
5.3.2	Major System Design Capacity	5.6
5.4	DRAINAGE SYSTEM COMPONENTS	5.7
5.4.1	Roof Leaders	5.7
5.4.2	Foundation Drains	5.7
5.4.3	Sump Pumps	5.8
5.4.4	Lot Drainage	5.9
5.4.5	Catch Basins	5.9
5.4.6	Pipe Systems and Outfalls	5.10
5.4.7	"Three Pipe" Systems	5.10
5.4.8	Roads	5.10
5.4.9	Gutters and Swales	5.11
		5.11

		Page
5.4.10	Receiving Waters	5.12
5.4.11	"Super-pipes"	5.12
5.4.12	Outfall Channels	5.12
5.5	RUNOFF CONTROL	5.13
5.5.1	Detention Ponds	5.14
5.5.2	Other Runoff Control Alternatives	5.21
6.0	**METHODS FOR QUANTITY ANALYSIS**	6.1
6.1	GENERAL	6.1
6.2	THE RAINFALL/RUNOFF PROCESS	6.2
6.3	RAINFALL CONSIDERATIONS	6.3
6.3.1	Antecedent Moisture Conditions	6.4
6.3.2	Intensity Duration Frequency Curves	6.5
6.3.3	Spatial Distribution of Rainstorms	6.9
6.3.4	Temporal Distribution of Rainstorms	6.10
6.3.5	Design Storm Duration	6.14
6.4	RUNOFF ESTIMATION METHODS	6.16
6.4.1	Rational Method	6.16
6.4.2	Isochrone Method	6.21
6.4.3	SCS Method	6.24
6.4.4	Deterministic Methods	6.25
6.4.5	Snowmelt	6.25
6.5	HYDROGRAPH ROUTING	6.27
6.5.1	Hydrologic Routing	6.27
6.5.2	Unit Hydrograph Methods	6.27
6.5.3	Hydraulic Routing	6.28
6.6	COMPUTER MODELS	6.30
6.6.1	Stormwater Management Model (SWMM)	6.31
6.6.2	Hydrograph Volume Method (HVM)	6.37
6.6.3	Illinois Urban Drainage Area Simulator (ILLUDAS)	6.40

		Page
6.6.4	Hydrologic Model (HYMO)	6.41
6.6.5	Hydrologic Engineering Centre Programs	6.43
6.6.6	Model Calibration and Verification	6.46
7.0	**WATER QUALITY CONSIDERATIONS**	
7.1	GENERAL	7.1
7.2	WATER QUALITY CHARACTERISTICS OF STORMWATER	7.1
7.3	THE NEED FOR WATER QUALITY ANALYSIS	7.2
7.4	METHODS OF WATER QUALITY ANALYSIS	7.4
7.4.1	Sedimentation	7.5
7.4.2	Water Quality Simulation	7.5
7.5	DETENTION PONDS AS TREATMENT FACILITIES	7.6
7.6	OTHER METHODS OF WATER QUALITY CONTROL	7.10
7.7	EROSION AND SEDIMENT CONTROL	7.11
		7.12
8.0	**OPERATION AND MAINTENANCE CONSIDERATIONS**	
8.1	MINOR SYSTEM	8.1
8.2	DETENTION FACILITIES	8.1
8.2.1	General	8.3
8.2.2	Equipment Access	8.3
8.2.3	Turf and Landscape	8.4
8.2.4	Aquatic Weed and Algae Control	8.4
8.2.5	Mosquito Control	8.6
8.2.6	Aeration and Circulation	8.7
8.2.7	Makeup Water	8.8
8.2.8	Winter Activities	8.8
8.2.9	Sediment Control	8.9
8.2.10	Monitoring	8.10
8.2.11	Operation and Maintenance Costs	8.13
8.3	PUMPING FACILITIES	8.15
		8.16
REFERENCES		
APPENDIX A	**EXAMPLE OF TYPICAL STORMWATER MANAGEMENT DESIGN STANDARDS**	

LIST OF TABLES

	Page
TABLE 4.1 TYPICAL DESIGN RETURN PERIODS FOR BRIDGES AND CULVERTS	4.3
TABLE 5.1 PERMISSIBLE DEPTHS FOR SUBMERGED OBJECTS	5.12
TABLE 6.1 ANTECEDENT MOISTURE DATA - ALBERTA	6.6
TABLE 6.2 ATMOSPHERIC ENVIRONMENT SERVICES RAIN GAUGE STATIONS--RAINFALL INTENSITY-DURATION-FREQUENCY DATA FOR ALBERTA	6.8
TABLE 6.3 TYPICAL URBAN RUNOFF COEFFICIENTS FOR 5 TO 10 YEAR STORMS	6.18
TABLE 6.4 SELECTED RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS	6.19
TABLE 7.1 URBAN RUNOFF CHARACTERISTICS	7.4
TABLE 8.1 RECOMMENDED QUALITY LEVELS FOR DETENTION PONDS	8.14

LIST OF FIGURES

		Page
FIGURE 3.1	ALBERTA ENVIRONMENT APPROVAL OF STORMWATER MANAGEMENT PROJECTS UNDER THE CLEAN WATER AND WATER RESOURCES ACTS	3.7
FIGURE 4.1	WATERSHED PLANNING AREA	4.10
FIGURE 5.1	MAJOR/MINOR SYSTEMS	5.2
FIGURE 5.2	TYPICAL POND CONFIGURATIONS	5.17
FIGURE 6.1	TYPICAL I.D.F. CURVE FORMAT	6.7
FIGURE 6.2	DESIGN STORM DISTRIBUTIONS - 5 YEAR RETURN PERIOD EVENT - EDMONTON	6.13
FIGURE 6.3	STORAGE BY RATIONAL METHOD	6.22
FIGURE 6.4	ISOCHRONE METHOD	6.23
FIGURE 8.1	ICE FAILURE/LOAD OBSERVATIONS	8.11

1.0 INTRODUCTION

Over the years, the population trend in Alberta has been towards increased urbanization; at the time of the 1981 Census, 77% of the Province's population lived in urban centres. Both this increasing degree of urbanization and the associated higher public expectations for runoff control have been the underlying forces in the trend toward the increasing use of stormwater management principles. The stormwater management philosophy has introduced the concept of dual drainage (major/minor) system analysis and the use of stormwater detention facilities for both peak flow rate and water quality control. It fostered the development of scientific methods which have displaced the traditional use of empirical formulae such as the Rational Method for the analysis of complex drainage systems. These changes have evolved since the middle 1960's, aided by the considerable amount of research in the United States funded by the U.S. Environmental Protection Agency. In Alberta, the rapid growth in the amount of urban development taking place during the 1970's forced developers, consultants and the larger urban communities to reassess drainage system design practices and standards.

A high quality of stormwater management is a necessary ingredient for orderly municipal growth. Stormwater management usually requires the use of surface facilities to store or convey runoff from extreme rainfall events. These facilities must often compete for space with other servicing components and land uses in a development. As a result, the planning of a stormwater management system must conform to the general development plan of the municipality which it serves.

There has been a tendency in the past to consider stormwater runoff as a liability. In the modern planning process, however, the potential for the beneficial use of stormwater should also be considered. Schemes for stormwater management facilities should consider multi-purpose applications wherever possible. Some stormwater management facilities can be aesthetic and recreational amenities for the communities in which they are located.

The development of stormwater management schemes leads to a need for the consideration of stormwater quality with respect to street cleaning and de-icing, storm sewer system cleaning, solid wastes collection, and control of erosion from construction sites. In addition, facilities not specifically designed for drainage such as parking areas, public parks and rooftops can be used as integral components of an overall plan to control peak stormwater runoff. Proper consideration and implementation of this positive approach, in conjunction with good policy and planning, will ensure an adequate standard of stormwater management in the future.

The purpose of this report is to discuss aspects of the planning, analysis, design, construction, operation and maintenance of stormwater management systems that are relevant to Alberta. In the past fifteen years there has been an abundance of literature written on the subject of stormwater management, a considerable portion of which was reviewed in the preparation of this document (a Bibliography of over 200 relevant papers and books has been included). In the subsequent seven sections, various aspects of stormwater management are discussed, particularly as they relate to application in Alberta:

Section 2 History

- A history of the evolution of stormwater management is presented for the several regions in Canada which have experienced the need for a more comprehensive approach to urban drainage.

Section 3 Legislation

- Requirements of the Clean Water Act and the Water Resources Act for implementing stormwater management systems are reviewed.

Section 4 Planning

- Planning for stormwater management is discussed. Four levels of activity are outlined. These consider a drainage system from the most general perspective, in the context of the overall watershed, down to considerations for the implementation of facilities.

Section 5 System Components

- The important concept of the dual drainage (major/minor) system is presented. The components of the minor and major drainage systems are described. The behaviour and the levels of service provided by each component are discussed.

Section 6 Rainfall/Runoff

- Rainfall considerations and methods for calculating flows in stormwater management systems are described. The methods range from the Rational Method and other hand calculation procedures to the most complex computer simulation models (many of which are now operable on micro computers).

Section 7 Water Quality

- Concepts for water quality analysis and modelling are discussed. The water quality characteristics of stormwater and the importance of sediment control are identified. Several water quality models are described to assist in determining where their application might be warranted.

Section 8 Operation and Maintenance

- Operation and maintenance considerations for stormwater management systems have not been well documented in the past. Various aspects related to the operation and maintenance of minor system components, detention facilities and pumping facilities are discussed.

Each section has been written as a self contained discussion. For those wishing to pursue a subject in more detail, references made in the text to articles in the Bibliography should point the way. For those with more pragmatic interests, an appended document entitled "Example of Typical Municipal Stormwater Management Design Standards" summarizes recommended practice in the Province of Alberta.

2.0 STORMWATER MANAGEMENT IN PERSPECTIVE

The term "stormwater management" first came into use in the early 1960's to signify a new approach to the drainage of urban land - an approach which considered impacts on receiving waters and sought to provide a greater level of service to the public. The history of stormwater management and the experience of its application in several regions of Canada are discussed in the following sections to provide a perspective on present practices. Three major points emerge from this discussion:

a) Urban development generally results in a large increase in both the volume and rate of runoff which reaches receiving watercourses.

b) Control of this increased runoff is often required in order to reduce the impact on the receiving waters to an acceptable level.

c) Drainage systems must consider the runoff from both small frequent events and large infrequent events.

2.1 THE EVOLUTION OF STORMWATER MANAGEMENT

Drainage Objectives

The objective of urban drainage has always been to quickly and efficiently drain the runoff from frequent rains and snowmelt events away from developed areas as a convenience to property owners. The earliest works constructed to provide this convenience were open channels, drainage gutters and ditches. Owners of property along such open drainage channels could observe flows on a frequent basis, and since the runoff from both small and large storms followed the same route, had some expectation of what areas might flood during large storms.

As the quality of urban developments improved, there was an increased demand for servicing convenience, and underground pipes replaced open channels in many

areas. Because pipe systems were more expensive to construct, their capacities were limited to that required to transport the more frequent rainstorms. Convenience drainage was much improved, since runoff would travel only a short distance overland before disappearing to the underground system.

As the size and number of urban developments increased, several problems with this "traditional" urban drainage approach became evident. Serious flooding and erosion was occurring on some of the watercourses into which the pipe systems discharged. Also, damaging localized flooding was occurring during infrequent but large rainstorms which exceeded the capacity of the underground drainage system. Runoff from these large events, which had previously followed the open drainage channels, now ponded in unplanned and usually undesirable locations. Where catchbasins became clogged at low spots, runoff ponded until the blockages could be removed. In some areas, the overloading of the underground pipes caused water to back up into foundation drains, often flooding basements.

A New Approach to Urban Drainage

The increasing intensity of urban development and growing concern for these runoff related problems resulted in a rethinking of urban drainage practices in the late 1960's. The concept of "systems" in engineering was becoming popular at about that time. This led to the principle that urban drainage should be treated as a system with consideration given to its behaviour under both frequent and infrequent events, and to its effects on downstream areas.

In order to control the flooding and erosion problems, various methods of temporarily storing urban runoff were adopted. The most common was the stormwater pond which receives urban drainage and releases it at a controlled rate. The problem of major storms which exceed the capacity of the underground system was dealt with through the recognition that there are two drainage routes to consider - a minor system consisting of pipes or channels capable of transporting runoff from frequent storms, and a major system capable of storing and/or transporting runoff which exceeds the capacity of the minor system.

The rapid growth in the availability of digital computers in the 1960's and 1970's led to the development of many computer simulation models for the analysis of urban drainage. The use of these models has become an important aspect of stormwater management.

The following section briefly describes the experience of the application of stormwater management to developments in several regions across Canada, including Alberta.

2.2 THE ONTARIO EXPERIENCE

Stormwater management practices evolved rapidly in Ontario because of the high population and intensity of development in the southern part of the province. Consideration of the Ontario experience is useful because of the large number of existing stormwater facilities. However, it is also important to recognize the differences in climatic and drainage characteristics in order to avoid copying techniques which may not be suitable for conditions in Alberta. This must be considered in reviewing the experiences in other regions as well.

Ontario Conditions and Requirements

The most intensive urban development in Ontario started in Toronto and spread north through Richmond Hill, east to Oshawa and west through Oakville to Hamilton. At most locations, development started at or near Lake Ontario and proceeded north up the watersheds. This placed steadily increasing pressure on the valley systems as erosion and flooding problems accelerated. Many older communities located downstream near the lake have substantial areas of development in the floodplain, making the control of flooding even more important.

Because of the emphasis on downstream flooding, most stormwater management facilities have been designed on the principle of restricting post-development flow peaks to pre-development rates, usually for the 2 year to 100 year events.

The allowable release rates are sometimes varied depending on the nature of downstream flooding problems, most of which have been extensively studied. Varying stormwater management requirements in this manner avoids the construction of unnecessary, or unnecessarily large, storage facilities. It illustrates the need to match the objectives of stormwater management facilities to the characteristics of the receiving streams. In some instances frequent events may be controlled while larger events are allowed to be discharged without control.

Stormwater Management in Ontario

Water management in Ontario is primarily the responsibility of two provincial government ministries (Natural Resources and Environment) and thirty-eight Conservation Authorities. The boundaries of the Conservation Authorities are coincident with watershed boundaries. The Conservation Authorities and the municipalities are the main approving agencies for stormwater management facilities. The required approval of the Conservation Authorities ensures that the agency which deals with flooding and erosion problems on a watershed basis also has control over the type of stormwater management which is required. Water resources management on a small watershed basis has been effective in Ontario.

The IMPSWM Group

As the nature of stormwater management facilities evolved, so did the analytical methods used to design them. The approving agencies became swamped with alternative methods for calculating flows and storage volumes, and there were at times quite different estimates of flow for the same area. This left the approval agencies in a quandry over which estimate to accept.

In 1978, Dr. Paul Wisner of the University of Ottawa formed a group called IMPSWM (for IMPlementation of StormWater Management). The purpose of this group, consisting of consultants, academics and individuals from government, was to identify needs in the design of stormwater management systems so they

could be resolved in a co-operative forum. IMPSWM published a series of papers dealing mainly with computational problems in urban hydrology, and revised and upgraded commonly used models, thereby helping to standardize urban hydrology simulation.

Master Drainage Plans and Drainage Manuals

Many municipalities began preparing Master Drainage Plans which examined watersheds in an integrated manner, identifying preliminary locations and criteria for stormwater facilities. Cooperation with the Conservation Authorities ensured that concerns beyond the boundaries of the individual municipalities were considered.

Detailed Drainage Design Manuals were prepared by many municipalities providing both a rationale for stormwater management and a handbook of data and information for the design of facilities. The municipalities of Scarborough, Markham, Oakville and Barrie are examples of communities with such manuals. These manuals, in conjunction with the master drainage planning which has been undertaken, have done a great deal to improve the quality of urban drainage in the municipalities they cover. Consultants, developers and approval agencies are able to work together much more effectively when such work has been undertaken. It is particularly important for the development industry to have the ground rules for development clearly and explicitly stated.

Stormwater Facilities in Ontario

Hundreds of stormwater management facilities have been constructed in Ontario, most for the purpose of regulating peak flows. Both wet and dry ponds are in common use. Underground stormwater tanks, rooftop and parking lot storage, and other methods of stormwater control have also found application.

2.3 THE WINNIPEG EXPERIENCE

The Winnipeg influence on Canadian stormwater management practices stems from its early and extensive use of stormwater storage lakes.

Physiography

Located in south central Manitoba, Winnipeg is built around the confluence of the Red and Assiniboine rivers. Large areas of the City lie within the natural floodplain of the Red River. The importance of river transport to early commercial activity resulted in the initial urban growth along the river banks. The topography is extremely flat with ground levels rarely greater than 10 metres above the normal summer water level of the river.

Flooding Problems

Flooding has been a continuing problem throughout the City's history. This is both from the rivers (which frequently attempt to use their natural floodplains) and from the sewer systems (where overloading causes water to be stored in basements, underpasses and other manmade depressions).

The basic defence against river flooding is provided by a system of primary dykes along the Red River. As basement floor elevations are rarely much higher than the dyke crest (many are below it), all sewer outlets penetrating the dykes are gated. Most of the sewer outlets are also equipped with flood pumping stations which operate when the river levels are higher than the outlets.

In addition to the dyking system, a high capacity floodway diverts a large portion of the Red River flow around the City, while an upstream impoundment and diversion channel control the flows of the Assiniboine River.

Sewer Services

Most of the older areas of the City are drained by combined sewer systems, which convey both runoff waters and domestic wastewater in a common pipe. Initial construction of the system started around the turn of the century. Most of the original brick lined tunnels are still in service today. Over time, the combined sewer systems were expanded as development moved away from the rivers. The original design criteria and design methodologies for these systems were not documented, but it is evident that some expansion was anticipated. Design criteria for the more recently constructed combined sewer systems required them to be capable of discharging a rainstorm with a 1 in 10 year return period. Inspections of the older systems show that careful attention was paid to construction details, resulting in hydraulically efficient systems.

Separate stormwater and sanitary sewer systems first appeared in the early 1960's, often as extensions to the upper ends of some of the combined sewer systems. While sewer separation may have been introduced to combat basement flooding it is more likely that the initial impetus was to further extend the usefulness of the combined sewer systems. These had reached their physical limit in terms of sufficient depth to make sanitary connections.

The separate storm sewer extensions were designed by the Rational Method to discharge a 1 in 5 year event operating surcharged to the ground at the high end. As the intent was to extend the systems as far as possible, sewer grades are as flat as practical and the sewers are relatively large. Detailed analysis of some of these systems in recent years demonstrates that the resultant insystem storage contributes significantly to the level of service they presently provide for short duration storms.

In the late 1960's and early 1970's basement flooding in the combined sewers and in some of the newer separate sanitary sewer systems had grown to a problem of epidemic proportions. Several factors contributed to these problems, the most significant being an increase in the amount of impervious surfaces within the drainage basins.

Drainage Planning and Detention Ponds

By the late 1960's most of the remaining developable land in the Winnipeg metropolitan area was located several kilometers from the rivers with very limited access to them. This together with the ongoing flooding problems forced an examination of the drainage philosophy. This process was facilitated by the establishment of a single municipal jurisdiction in 1971 replacing the former metropolitan system. By the early 1970's drainage planning studies had been carried out for all of the undeveloped land within the City boundaries. It was evident that the concept of stormwater storage would be the only economically feasible solution to overcome the hydraulic limitations to storm drainage in most new developments.

In response to this pending proliferation of stormwater impoundments the City drew up a set of guidelines, "Stormwater Management by Use of Impoundments" (City of Winnipeg, 1974), to ensure a coordinated approach to the design of these facilities. This was followed by the "Drainage Criteria Manual for the City of Winnipeg" (J.F. MacLaren, 1974). To a large extent the guidelines drew upon the experience of five stormwater storage systems which were in operation at that time. Four of the systems are gravity discharged with permanent water bodies located in residential settings. It is somewhat ironic that the first of these systems, the six-lake Southdale complex, could have been economically serviced by a conventional storm sewer system. The lake system was implemented by the developer as a marketing device, which proved to be highly successful.

The major/minor drainage concept was formally introduced in the Drainage Criteria Manual. Because of the flat topography, overland flow does not occur to any significant extent. Street storage of excess runoff during major events was accepted as an inconvenience, but not a serious problem except in specific locations such as underpasses. Thus, the opportunity provided by stormwater lakes to be terminal points for the major drainage system routes was not initially exploited to any great extent.

During 1976 and 1977 the City, in collaboration with the Canada Mortgage and Housing Corporation (CMHC), carried out an "Evaluation of Stormwater Impoundments in Winnipeg" (Chambers and Tottle, 1978). The most significant findings of this study were related to water quality. While the efficiency of the impoundments as sediment traps had been expected, the overall water treatment potential had not been anticipated.

The complexity of drainage problems facing the engineering community in the early 1970's stimulated interest in the application of computer models. As there were few models available for analysis of urban hydrology, this led to experimentation with the development of 'in-house' models. This effort was largely abandoned when SWMM (Storm Water Management Model) became publicly available in 1974. In particular, with the ability of the Extended Transport Block (1975) to simulate surcharged pipeflow (a frequent operating condition for most Winnipeg sewers) SWMM was quickly adopted as state of the art technology.

Summary

The impetus for the use of stormwater impoundments in Winnipeg was the high cost of providing a conventional minor drainage system. The cost effectiveness of on site storage versus large and long outfall sewers was the most significant factor. This, together with extensive problems within the existing drainage systems, identified a need for comprehensive long range planning to meet ongoing and future drainage requirements. This need was met by developing guidelines, design criteria, acceptable methods of analyses, and master drainage plans.

2.4 THE BRITISH COLUMBIA EXPERIENCE

Beginnings of Stormwater Management

The general increase in the study of urban runoff which occurred in the 1960's in the United States went unnoticed in British Columbia until the early 1970's. The first thorough examination of new stormwater management practices was conducted by the Greater Vancouver Sewerage and Drainage District (GVS & DD) in 1973. The GVS & DD comprises 17 communities covering about 1800km^2 in the Lower Mainland with a population of about 1,100,000. This is about 40% of the Province's total population. It has the authority to construct, finance and maintain all trunk and interceptor sewers, outfalls and watercourses in the area of the member municipalities.

Between 1953 and 1980 (approximately), most of the planning and construction for water, sewer and drainage systems in the Greater Vancouver Sewerage and Drainage District was based on the District's Rawn Report. This report, published in 1953, had a major influence on drainage philosophies for more than two decades.

Becoming aware of the new trends in stormwater management, the Greater Vancouver Sewerage and Drainage District initiated a study on stormwater management in 1973. The General Hydrological Study was completed in 1977. In the study, GVS & DD (1977a, 1977b) reviewed existing drainage design practices based on a questionnaire circulated to member municipalities and local consultants. They found that the majority of municipalities and consultants used the Rational Method as fostered by the Rawn Report (but with trepidation).

Respondents to the questionnaire indicated that they were in favour of using new approaches for urban drainage design. It was perceived that the more complex problems that existed in heavily developed urban areas could best be assessed using the computer models which were becoming available. To assist in the transition to computer analysis of stormwater systems, a history and state of the art review on storm runoff modelling was also presented. GVS & DD

participated in the City of Vancouver's West End Demonstration Project which was the first application of a computer model (Dorsch Consult's HVM model) in 1974. This was a planning study where the means for separating the combined sewers in the City's West End were identified. Later, a comparison of the HVM with the SWMM and ILLUDAS models was made (GVS & DD, 1975).

Rainfall Record Processing

GVS & DD also investigated the rainfall data acquisition procedures in operation, assembled historical data on computer tape, and outlined options for future data acquisition and processing. In the Lower Mainland, two types of rainfall events are typical of the region:

a) the steady, heavy rainfall events (synoptic scale cyclonic circulation events) that occur in the winter, and

b) the summertime cloudburst events (convective shower events).

The former is the source of the majority of urban flooding problems for the Lower Mainland. This is one of the fundamental reasons for differing stormwater management practices in the two provinces. Rainfall conditions in Alberta are dramatically different from those in the Lower Mainland. In Alberta, convective summer rain fall events are of predominant importance.

Fisheries Concerns

Another reason for differing stormwater management practices is related to the significant fisheries resource in British Columbia. Certain small creeks which have significant spawning beds are designated to receive no stormwater discharge. This concern to preclude any silt discharge to these streamcourses results in stormwater management schemes which include diversions of minor system flows to other more favourable discharge points.

Detention Facilities

Stormwater detention facilities are not an effective means of runoff control in Greater Vancouver because of the steady, long-duration nature of rainfall events. The one hour depth of rainfall is about 15% of the 24 hour depth whereas in Alberta, where detention ponding is more effective, this ratio is about 40%. GVS & DD (1977b) conducted some hypothetical sizing and costing exercises and could not find value in their application in the Lower Mainland. Presently, in British Columbia (with the exception of the District of Surrey), there are only about six existing stormwater detention facilities located in Nanaimo, Delta, Coquitlam and Kamloops.

The District of Surrey has adopted a concept of zero increase in peak runoff rate and the analysis of the dual drainage system as the basis for their stormwater management planning. Their policy is that on-site or off-site storage must be provided "to restrict the post-development peak runoff to the predevelopment grassland condition for five year return flows". They outline detailed procedures (either the Rational Method or the Surrey's own RUNOFF-I computer program) to determine the design flows and the amount of storage. As a result of this policy, there are presently one wet pond and 108 small dry detention facilities, located in the District of Surrey (with a 1981 population of about 147,000). This appears to be an unusual application of stormwater management for single event flow control which would not be of practical application in Alberta.

Water Quality Control

The First Flush concept was studied in detail for the first time in British Columbia during the period 1982 to 1985. This was one of the development considerations for B.C. Place, a large industrial/commercial development located on False Creek in Vancouver. False Creek is an embayment on Burrard Inlet, which as a result of B.C. Place and the related high density residential development, is experiencing higher expectations for its water quality. Plans for facilities to separate and store the first flush of stormwater runoff (for later pumping to the sanitary sewer system) were incorporated into the design of B.C.

Place. As False Creek is also the recipient of combined sewer overflow discharges, the Firsh Flush facilities will likely not be implemented until the combined sewer overflows (which have more severe consequences) are eliminated. GVS & DD and the City of Vancouver are presently studying the means to eliminate these overflows.

Stormwater Management in the Interior

In the interior of British Columbia rainfall is less intense than on the Prairies but is similar in its seasonal distribution. This, coupled with steeper terrain, less intense development and more pervious soils results in minor system works which are of a much smaller size than for similarly sized centres in Alberta. As a result, stormwater management did not gain popularity in the Interior until about 1982 when a master drainage plan for the Guerin Creek watershed was prepared for the City of Kamloops using the ILLUDAS model. Kamloops and Penticton have since had other studies carried out by consultants. Stormwater management practice in Kelowna is somewhat unique as extensive use is made of infiltration wells for the disposal of stormwater to the surficial aquifer.

Detention ponding could be of value for stormwater management in the Interior of British Columbia (Hodgson and Johnson, 1985), where the ratio of the short to long-duration rainfall depths are in the range of 40% to 50%. However, the typically steep slopes make the construction of larger, more cost effective ponds difficult. The pervious soils also result in the need for a greater geotechnical engineering input for the design of liners for the detention facilities.

2.5 THE ALBERTA EXPERIENCE

Between 1971 and 1981 the total population of Alberta grew by 37% to about 2.3 million people. About 77% of the present population lives in urban centres. Although there are seven centres with populations ranging from 25,000 to 60,000, the majority of the urban population lives in Edmonton (560,000) and Calgary (621,000). It is the rapid growth of the 1970's, particularly in these two major

cities, that gave impetus to the use of stormwater management concepts in the planning and implementation of drainage facilities for new developments.

City of Edmonton

Prior to the 1970's, most of the City of Edmonton was drained by combined sewer systems. Rapid expansion started in 1971 when 61 km2 of land was annexed for the areas of Castle Downs, the Lake District and Mill Woods. In the following year 26 km2 of West Jasper Place was annexed.

The City's Water and Sanitation Department began a systematic utilization of stormwater management methods to meet the needs of these new development areas. Runoff control was provided in different ways in different parts of the City. In the north, the Lake District and Castle Downs utilized detention storage to limit the load that the outflow from these new areas might place on the existing facilities between them and the North Saskatchewan River. In the South, a large diameter (up to 5200 mm) tunnel was constructed to serve as the drainage trunk for Mill Woods and other areas.

Through this rapid growth period the City developed a drainage planning and implementation procedure that includes watershed drainage planning and master drainage plans. Annexations to Edmonton culminated in 1982 when an additional 370 km2 became the City's responsiblity (bringing the total area to 700 km^2). The City engaged consultants to develop Watershed Plans to properly consider the drainage requirements of the lands it had annexed. Computer modelling, major/minor drainage system analyses, and environmental impact assessment were activities that consultants had to carry out in the development of these plans.

The City has been using simulation modelling since 1974, initially as part of an ongoing relief and separation program for the combined sewers and subsequently for stormwater and sanitary system planning. A considerable amount of monitoring and calibration has been done in conjunction with this work. The City has developed a comprehensive rainfall and sewer flow data acquisition and

processing system which monitors conditions at 37 permanent (16 rain and 21 flow) stations and a number of temporary locations.

City of Calgary

The City of Calgary's major expansion took place in 1961 when 195 km^2 of land was annexed. Since then, another 137 km^2 has been added through numerous other annexations (bringing the total area to 535 km^2). Calgary's drainage facilities have evolved making use of the Bow River, Elbow River, Nose Creek, Beddington Creek and the Western Irrigation District's main canal as receiving streams for stormwater flows. Fish Creek, in the south, is part of a Provincial Park and the discharge of stormwater to it is restricted. The City has a rainfall monitoring network to measure both temporal and spatial variations of storm events. During the 1970's, the City conducted a number of planning studies to determine options for drainage of newly annexed and other undeveloped lands.

Calgary's 68 Street S.E. Retention Lake, with a 20 ha surface area, is the largest stormwater management facility in the Province. Stormwater reaches this facility via a trunk sewer which intercepts the original sewer system in the northeast corner of the City and conveys it around the intervening development. This allowed further expansion of sewer systems to the north.

Detention Facilities

During the 1970's, the use of detention ponds became common practice in Alberta. There are 3 wet ponds and 2 dry ponds in the Calgary region. In the Edmonton region there are presently about 25 wet ponds and 8 dry ponds. One of the primary reasons for constructing detention ponds in the Province is the lack of a receiving stream with sufficient capacity to discharge uncontrolled urban

runoff. Several such facilities are located in smaller communities including Sherwood Park, Lloydminster and Morinville. Runoff control to lessen peak discharges has also been beneficial in mitigating potential erosion problems. In addition, wet ponds have provided aesthetic benefits in new developments; these lake-like facilities have been used to market developments in their vicinity.

3.0 APPROVAL CONSIDERATIONS

3.1 ROLE OF ALBERTA ENVIRONMENT

By establishing guidelines and policies, Alberta Environment is able to provide an impetus for good stormwater management practices across Alberta. Recognizing that good stormwater management can be achieved only through cooperation with municipalities, developers, and consultants, it is the role of Alberta Environment to promote the protection, while also enhancing the environment and recreational potential, of the watersheds under its jurisdiction.

3.2 OBJECTIVES OF ALBERTA ENVIRONMENT

The following objectives have been developed in order to achieve the specified role of Alberta Environment in controlling the development of stormwater management systems:

a) Minimize the potential for flooding and erosion downstream of the development.

b) Ensure that the hydraulic capacities of existing watercourses through the development area are not adversely affected, either upstream or downstream of the development.

c) Ensure that the developed areas are located above the 1:100 year flood level.

d) Inclusion of runoff control through storage, with a maximum release rate equal to the predevelopment rate for flood events up to and including the 100 year event (in some cases, more stringent criteria may be necessary), except when it can be proven that increased runoff volumes and rates from all future upstream development will not cause physical or environmental damage downstream.

e) Reduce, to acceptable levels, the potential risk of health hazards, loss of life and property damage from flooding within the development.

f) Reduce, to acceptable levels, the inconvenience caused by surface ponding and flooding within the development.

g) Ensure that proposed designs are integrated with and conform to approved master drainage plans for the development area.

h) Minimize the impact of any changes in the groundwater regime, the erosion rates or the sediment transport rates caused by the development, especially during construction.

i) Maintain the natural stream geometry, insofar as it is feasible, in achieving stormwater management objectives.

j) Reduce the contaminant loadings from urban stormwater discharges through the use of design techniques such as detention storage.

It is also the objective of Alberta Environment to increase public awareness of the benefits of good stormwater management policies, and to work effectively with municipalities, developers and consultants to coordinate these policies with the general development plans across the province.

3.3 SUBMISSION REQUIREMENTS

3.3.1 Clean Water Act

Pursuant to the Clean Water Act and Clean Water (Municipal Plants) Regulations, no person shall construct or extend a municipal sewer (being defined herein as a stormwater management project) without first applying and receiving a Permit to Construct from the Director of Standards and Approvals. The

application for the Permit shall be made by either the owner or his authorized agent, and shall include the following information:

a) An engineering report establishing the need and feasibility of the proposed project. The report should outline the methodology of flow estimation.

b) A location plan showing the proposed stormwater management works including detention/retention ponds, catchment areas, watercourse improvements or relocations, and the receiving watercourse or waterbody.

c) Detailed plans and specifications of the proposed sewer and/or stormwater management systems, stamped and signed by a professional engineer licenced to practice in Alberta. Refer to section 3.3.2 (b) for detailed drawing requirements.

d) A detailed cost estimate.

e) Any additional information as required by the Director of Standards and Approvals.

The application is typically made in two steps. First, a suitable engineering report and plan (items a and b) are submitted with an Application for Permit. After review by Alberta Environment a conditional Permit to Construct will be issued. Following detailed design, the plans, specifications and other information (items c, d and e) are submitted for review prior to the issuance of the Final Construction approval.

3.3.2 Water Resources Act

Pursuant to the Water Resources Act, a Licence is required to impound water for the purpose of water management, flood control, erosion control, flow regulation, construction, recreation, or the propogation of fish or wildlife. A

Licence is also required to divert surface water (by a method other than impoundment) for the purpose of water management, flood control, drainage, erosion control, or channel re-alignment. Therefore, a Licence is required for stormwater management projects which involve impoundment by retention or detention storage, diversion of watercourses, or downstream watercourse improvements to provide an adequate outlet for a development.

Prior to the construction of any stormwater management project requiring a Licence, the proponent shall make application and obtain the necessary approvals under the Water Resources Act. The submission should be made to the appropriate Regional Office of the Water Resources Administration Division of Alberta Environment, and shall contain the following information:

a) A completed Application for Licence form, together with the detailed plans of the proposed works and other supporting documentation. This application for Licence shall be signed by a responsible officer of the applicant pursuant to the provisions of the Water Resources Act.

b) One (1) complete set of sepia or mylar (or other similar type of transparency) drawings, and three (3) sets of paper copies. All drawings shall bear the stamp and signature of a professional engineer registered with the Association of Professional Engineers, Geologists and Geophysicists of Alberta. Location plans shall be clearly referenced to the legal land location system in terms of delineating the section and quarter section boundaries. Cross-section drawings of detention/retention facilities should indicate elevations of inlet and outlet inverts and design water levels. Also, profiles and cross-sections of existing watercourses and proposed modifications should be included if watercourse changes are proposed.

c) A location plan showing the proposed stormwater management works including detention/retention ponds, catchment areas, watercourse improvements or relocations, and the receiving watercourse or waterbody.

d) Hydrologic calculations and design methodologies used in the design of the stormwater management works (existing information pertaining to hydrologic data can be obtained from the Hydrology Branch, Technical Services Division of Alberta Environment, 427-6810).

The application is made after the completion of the final design and prior to construction. Construction can commence after the issuance of an Interim Licence. Upon a satisfactory inspection, after construction completion, a Final Licence will be issued.

3.3.3 Approval of the Local Authority

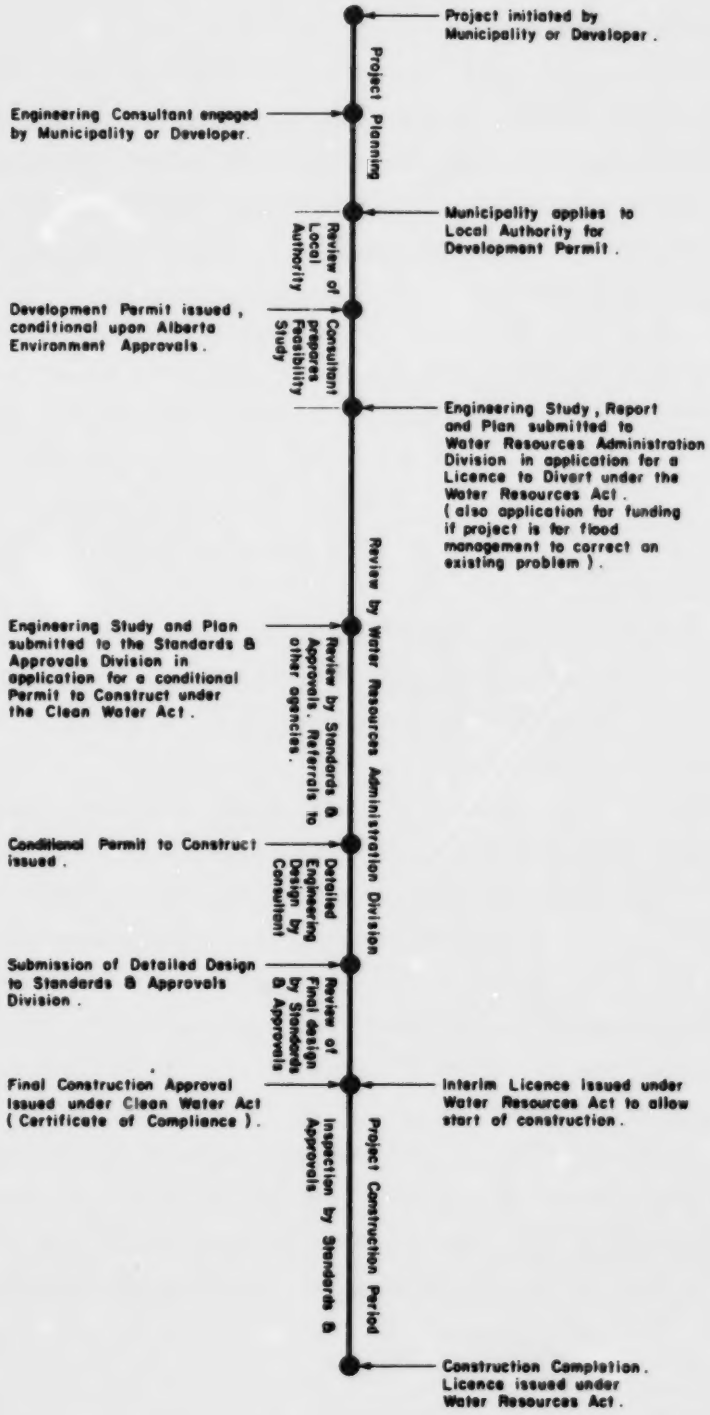
There may be certain instances where the applicant shall be required to submit the written approval of the Local Authority before the issuance of a Permit to Construct can be considered. These instances are as follows:

a) Where application for a Permit to Construct for any storm drainage project within the boundary of a Local Authority is made by a person other than the Local Authority (municipality) or the municipality's authorized agent (consulting engineer); or,

b) Where the Director of Standards and Approvals requests a clarification of the Development Permit from either a County, Municipal District, or Improvement District, for any municipal stormwater management project within the boundary of the particular rural municipality. This may be required prior to issuance of a conditional Permit to Construct in order to clarify any land use conflicts associated with the project.

3.3.4 Approval Coordination

Figure 3.1 illustrates the step-by-step procedure required in coordinating the approval of a municipal stormwater management project in Alberta. The issuance of a conditional Permit to Construct under the Clean Water Act (see Section 3.3.1) and/or the Water Resources Act does not imply or represent approval by any other government agency or municipal authority.


Project initiated by Municipality or Developer.
Project Planning
Engineering Consultant engaged by Municipality or Developer.
Review of Local Authority
Municipality applies to Local Authority for Development Permit.
Development Permit issued, conditional upon Alberta Environment Approvals.
Consultant prepares Feasibility Study
Engineering Study, Report and Plan submitted to Water Resources Administration Division in application for a Licence to Divert under the Water Resources Act. (also application for funding if project is for flood management to correct an existing problem).
Review by Water Resources Administration Division
Engineering Study and Plan submitted to the Standards & Approvals Division in application for a conditional Permit to Construct under the Clean Water Act.
Review by Standards & Approvals. Referrals to other agencies.
Conditional Permit to Construct issued.
Detailed Engineering Design by Consultant
Submission of Detailed Design to Standards & Approvals Division.
Review of Final design by Standards & Approvals
Final Construction Approval issued under Clean Water Act (Certificate of Compliance).
Interim Licence issued under Water Resources Act to allow start of construction.
Inspection by Standards & Approvals
Project Construction Period
Construction Completion. Licence issued under Water Resources Act.

ALBERTA ENVIRONMENT APPROVAL OF STORMWATER MANAGEMENT PROJECTS UNDER THE CLEAN WATER AND WATER RESOURCES ACTS

FIGURE 3.1

4.0 PLANNING FOR STORMWATER MANAGEMENT

The term "stormwater management" implies a comprehensive approach to the planning, design, implementation and operation of stormwater drainage improvements. The purpose of the stormwater management approach is to develop effective drainage systems which balance the dual objectives of maximizing drainage efficiency and minimizing adverse environmental impacts.

This section outlines a basic framework for planning the development of stormwater management systems. It deals with rural and urban drainage system considerations, four levels of drainage planning (from river basin planning to implementation), and discusses the merits of design standards for stormwater facilities.

4.1 CONSIDERATIONS FOR RURAL DRAINAGE SYSTEMS

Few drainage systems in inhabited areas remain unchanged from their natural state. The development of agriculture and of transportation networks has resulted in modifications to the natural drainage system. These modifications to land use and drainage patterns can be the source of drainage problems in rural systems.

Rural drainage improvements in Alberta may be cost-shared between the Province and a Local Authority under a policy entitled Position Paper #5. This policy is administered by the Regional Services Branch, Water Resources Administration Division, Alberta Environment. Guidelines and criteria for funding and implementing such projects are established. Information regarding this program is available from Alberta Environment.

Rural and urban drainage are inter-related since both may contribute to the overall hydrology of a watershed. Financial assistance under Position Paper #5 may be approved for urban drainage facilities in two particular circumstances. The first is where upstream rural runoff causes or contributes to flooding in an existing urban development. The second is where an outlet channel must be

constructed beyond an urban boundary to reach a discharge location with adequate capacity.

The following discussion provides background for the planning of drainage systems for smaller communities which must interface with rural drainage systems.

4.1.1 Road Drainage

Most drainage improvements in rural areas relate to road drainage. Rural roads are designed to shed water into roadside ditches. In Alberta, these ditches are usually wide and shallow with flat grades, and in most cases they significantly attenuate runoff hydrographs. Most adverse impacts caused by the road system occur where they cross watercourses; ditches with steep grades approaching creeks and rivers are susceptible to erosion which may result in sedimentation problems in the stream. Road crossing structures often restrict the flow in minor watercourses, however, in such cases it is usually the road that suffers the most damage when an extreme runoff event occurs. It is interesting that the concept of level of service is firmly entrenched in highway and rural road design, especially in relation to stream crossings. Invariably the capacity of such crossings in terms of flow frequency is determined by the importance of the road and the type of crossing. In contrast, this appears to be a rare consideration in the design of urban roadways. Table 4.1 shows typical design return periods for bridges and culverts for various types of roadway.

4.1.2 Agricultural Drainage

Most rural runoff in Alberta and the other prairie provinces occurs in the spring as a result of snowmelt. In most cases, significant runoff from summer rainstorms is an unusual occurrence. Since water is vital to plant growth, particularly in the form of soil moisture retention, agricultural drainage systems are not designed to be hydraulically efficient with respect to the rate of surface water removal. The major emphasis is on positive drainage to ensure that standing water does not remain in low areas for extended periods.

TABLE 4.1

TYPICAL DESIGN RETURN PERIODS FOR BRIDGES AND CULVERTS

Road Classification	Return period (years)	
	Culverts with total span up to 6.0 m	Bridges and culverts with total span exceeding 6.0 m
Freeway	50	100
Urban arterial	50	100
Rural arterial	25	50
Collector	25	50
Urban local	25	50
Rural local	10	25

Notes:

1. Total span for this purpose is the sum of individual spans or diameters, measured parallel to the road in the case of a bridge, and normal to the longitudinal axis in the case of a culvert.

2. The return periods listed are for guidance only and should be modified in the following cases:

   a) if the flood hazard in the vicinity of the site is unusually severe;
   b) if the road classification is likely to be upgraded or downgraded after construction;
   c) if the road has an unusually low traffic volume.

3. Taken from Drainage Manual Vol. 1, RTAC (1982).

The most significant impacts of agriculture on rural drainage systems result from either the drainage of wetland areas or the conversion of woodlands to pasture or cropland. These significantly increase the amount of runoff and erosion. Where such undertakings are planned on a large scale, regional drainage planning may be warranted.

4.1.3 Urbanization in Rural Watersheds

Where urbanization occurs in an essentially rural watershed the changes that occur in the hydrologic regime are very significant to the downstream system. Specific changes which will usually occur are:

a) an overall increase in the annual volume of runoff,
b) a much faster rate of runoff for any given event,
c) summer rainfall events which can result in significant runoff from the urbanized areas, while little or no runoff comes from the rural portion of the basin.

The seasonal change in the runoff pattern is particularly important. Many rural drainage channels routinely overflow their banks and cause flooding of adjacent land during spring runoff. Usually this is not a problem provided that the excess water drains away quickly. In many cases, particularly in the upper reaches of a watershed, the primary drainage routes are not incised channels but broad overland flow routes active only during spring runoff. During the summer months these floodplains may be productive farmland where flooding would cause extensive economic damage.

The imposition of rigid flow regulation policies for rural drainage based on pre-development/post development concepts should be avoided. It is imperative that the required capacities of discharge channels be determined on the basis of hydraulic analysis. This should include analysis of both channel capacities and the restrictions due to stream crossings. It should not be assumed that a flow rate which occurs without problems during the spring would not cause problems during the summer. In cases where high capacity channels exist, flow regulation

may be unnecessary for summer storms. In addition, such regulation may retard spring runoff to the extent that it coincides with upstream peak flows resulting in higher discharge rates than would otherwise occur.

4.2 DRAINAGE CONSIDERATIONS FOR URBAN AREAS

4.2.1 The Impact of Development

From a drainage perspective the most dominant characteristic of the urban landscape is the high degree of impervious ground cover. Urban areas are also characterized by systematic surface grading intended to direct the flow of water. A secondary effect of grading is to ensure a rapid rate of runoff. These factors result in more significant changes to the hydrologic regime in comparison with changes due to drainage works in rural areas.

Runoff events occur on a routine basis in urban areas. Summer rainstorms which would produce little or no runoff in rural areas produce significant runoff in urban areas. The overall effect of large amounts of impervious land surface in urban areas is to dramatically increase the runoff volume and the rate of runoff from each rainfall event. In contrast to rural areas, the runoff produced by snowmelt events in urban areas rarely exceeds the drainage system capacity.

4.2.2 Convenience Drainage for Urban Areas

Traditionally, drainage planning for urban areas has focused on the inconvenience caused to human activity by rainfall. As a result, urban drainage systems are characterized by positive grading of land away from buildings to roadways which in turn are graded to ensure rapid flow of surface water to a point where it can be discharged to an underground sewer system. The sewer system conveys the water to a point where it can be discharged to a watercourse or water body. The hydraulic efficiency exhibited by such systems under normal circumstances is

primarily due to the storm sewer system. In addition to hydraulic efficiency, sewers do not use additional land and require a minimal amount of maintenance in comparison to other municipal services.

Storm sewer systems are generally considered to be expensive, and the fact that the larger more costly trunk sewer components have to be installed first results in a high initial or front end cost. Economic considerations effectively limit the hydraulic capacity which can be provided by storm sewer systems. It is common practice to design these systems to discharge the flow generated by rainstorms with a 1 in 5 year return period.

4.2.3 Flood Protection in Urban Areas

While storm sewer systems work effectively under normal circumstances, occasional runoff events will occur where the system has insufficient capacity to discharge in the normal manner. The impacts of the runoff rate exceeding the capacity of the system can vary from temporary accumulation of standing water around a street inlet to large volumes of water flowing overland, causing flooding of buildings and other facilities. Situations occur in some systems where water may flow out of manholes and inlets in the downstream parts of the system onto streets and adjacent property. Until recent times it appears that little or no consideration was given to what would happen when the system capacity was exceeded.

With the passage of time, it has become apparent that the degree of flood protection provided by urban drainage systems designed primarily for convenience is not adequate. Provision must also be made to control the excess runoff resulting from infrequent events.

4.2.4 Storage for Flood Protection

Stormwater storage has proved to be a flexible approach to mitigating many urban drainage problems. Storage facilities include underground vessels (superpipes), temporary flooding of open space areas (parks, parking lots, etc.),

and storage ponds. The most common form of storage is the stormwater detention pond. This facility has become synonymous with stormwater management.

The introduction of a storage element into a drainage system serves to attenuate the hydrograph flowing through it, in effect reducing the peak flow rate. The attenuation capability, more commonly referred to as routing effect, is inherent in all hydraulic systems which transport intermittent or varying flows. Routing effects are smaller for efficient hydraulic sections such as pipes than for less efficient hydraulic sections such as natural channels.

Buried storage has limited application because of the relatively high costs involved. Impoundment of stormwater in large surface depressions is cost effective in many cases, especially in newly developing areas where land can be made available. The location of an impoundment along a natural drainage route greatly facilitates the control of excess runoff during major events.

Most stormwater impoundments are classified as detention storage facilities. These facilities temporarily store water in excess of the downstream conveyance capacity. Detention storage facilities can be either normally 'dry' or retain a permanent 'wet' water body. As hydraulic facilities they function in an identical manner. Retention storage facilities store water for extended time periods and may rely entirely on evaporation and infiltration for ultimate disposal of runoff. Retention storage facilities are not commonly used in urban environments (note: others define detention and retention differently; using the above definitions, the majority of urban storage facilities are termed dry ponds or wet ponds).

4.3 Drainage Planning In Context of A Watershed

4.3.1 General

A drainage system always exists, whether the land is in its natural state or has undergone some degree of development. Virtually any modification of an

existing drainage system will modify the hydrology of the system. An increase in runoff from a site will increase the flood/frequency relationship in downstream channels or conduits.

Urbanization of a natural drainage basin can result in increases in runoff of 400 percent or more (Environment Canada, 1977b). Also, stormwater carries a variety of water contaminants which may accumulate and damage aquatic environments and/or restrict water usage to some degree. The impacts of urbanization vary depending upon both the scale and type of drainage improvements carried out and the sensitivity of the receiving stream environment related to each drainage system.

The benefits of drainage improvements tend to be readily apparent and to be localized to the site of the drainage improvements. Adverse environmental impacts generally occur downstream of the improvement site and are not always readily apparent. These impacts are of concern as they tend to be accumulative over both time and space. It would be impractical to evaluate the environmental impacts of all localized drainage improvements within a watershed on an individual basis. Local improvements should be made in the context of an overall plan for the entire watershed. As a result, a hierarchical approach to the planning and implementation of stormwater management is required.

Planning for stormwater management must be carried out on a scale that allows assessment of impacts throughout the natural drainage system. In Alberta, the major river basins are used as macro planning units. These units are too large to be practical units for drainage planning, however, they may identify some constraints for smaller planning units.

At the other end of the spectrum, most drainage improvements are designed and implemented to service residential or industrial subdivisions of a few hectares. Land development and land use planning units are primarily influenced by legal boundaries, land ownership, and political jurisdictions. These units rarely coincide with natural drainage boundaries, and are usually too small for stormwater management planning.

It is evident that intermediate levels for stormwater management planning are required. For discussion, it is convenient to consider the following four levels of planning and design of stormwater management systems: major river basin plans, watershed drainage plans, master drainage plans, and implementation.

Level 1 - River Basin Plan

River basin planning typically considers the major river basins in the province. River basin planning is basically concerned with the supply of and demand for water as a resource. The impact of urban land drainage on the hydrology of Alberta's major rivers is negligible for most practical purposes. Urban and industrial developments do, however, have a significant impact on the water quality of these rivers.

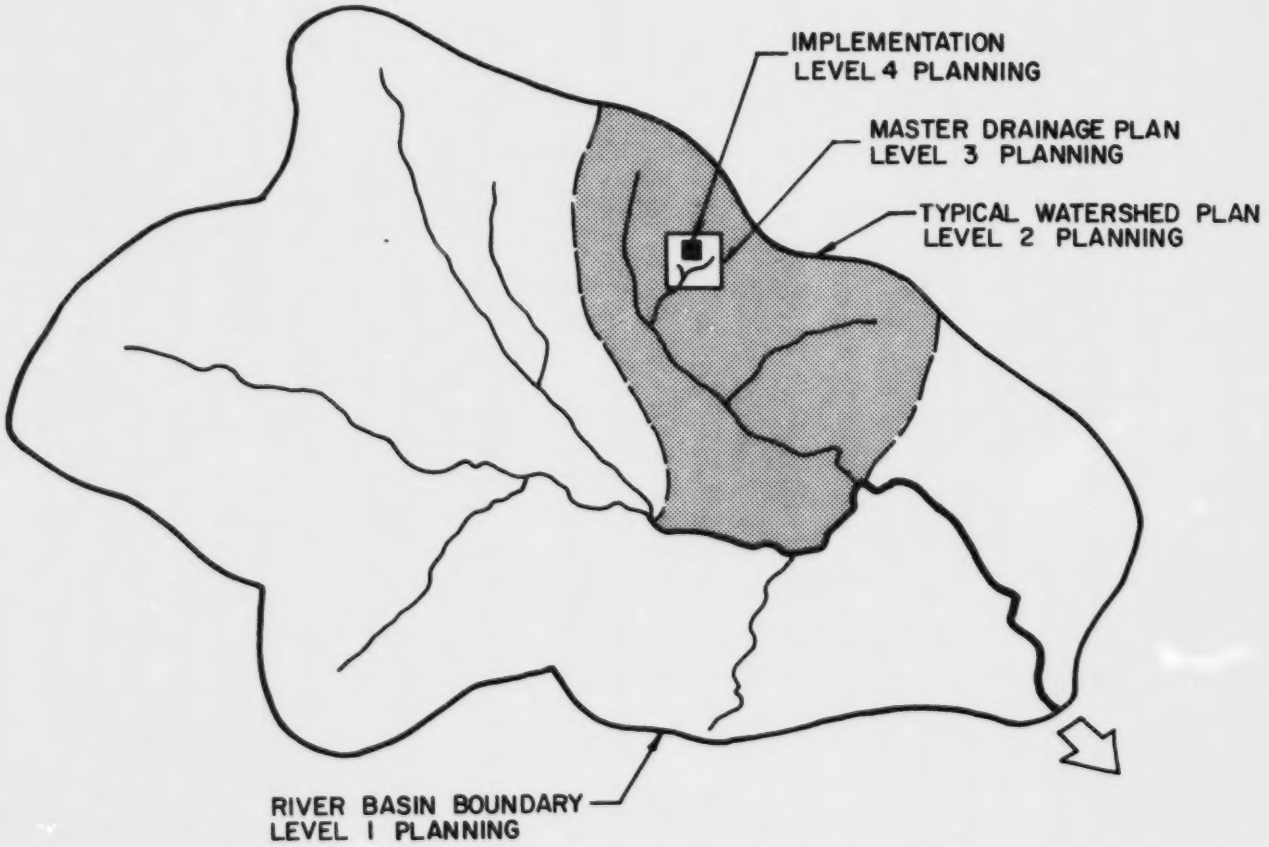
It is probable that urban storm drainage systems contribute a significant portion of the pollutant loads found in Alberta's rivers. It is possible that future restrictions on the quality of stormwater effluent may be imposed as a result of investigations at this level of planning. It may also be logical to assess the need for regional drainage plans at this planning level. In these cases boundaries for such regional studies would be defined based on anticipated development.

This level of planning is essentially a provincial responsibility.

Level 2 - Watershed Drainage Plan

Basic to the development of an effective stormwater management plan is the definition of areas within which the cause and effect relationships between individual drainage elements may be significant. A Watershed Drainage Plan should consider drainage requirements and potential impacts on a regional basis and develop a stormwater management plan at a conceptual level. A Watershed Drainage Plan therefore may consider the drainage requirements of a single drainage basin or a group of sub-basins which contribute to a point in the natural drainage system (Figure 4.1).

FIGURE 4.1



WATERSHED PLANNING AREA

Planning and implementation of storm drainage is traditionally a municipal government responsibility. Factors inhibiting development of Watershed Drainage Plans have been the initial study costs, coupled with few short term benefits. In many cases drainage basins cover more than one municipal administration.

Drainage problems created by development are not always visible in the early stages of development. The lack of comprehensive planning in such cases invariably leads to long term flooding and/or pollution problems. This has been demonstrated by many river systems in the United States. It is noteworthy that funding by the U.S. government for the relief of such problems is increasingly being made subject to the development of comprehensive stormwater management plans and administrative structures to implement them.

This level of planning likely requires joint participation of both provincial and municipal governments.

Level 3 - Master Drainage Plan

Master Drainage Plans should develop alternatives and identify optimal drainage solutions that conform with the objectives of the Watershed Drainage Plan and the realities of the proposed land use plan. The scope of the Master Drainage Plan generally covers a portion of the area served by the Watershed Drainage Plan such as one or more sub-basins. Ideally, it complements a "neighbourhood structure plan".

Although this level of planning is basically a municipal responsibility, the development of such a plan may be undertaken by (or in co-operation with) private developers with large land holdings within the plan boundaries.

Level 4 - Implementation

At this level, the detailed design and construction of drainage facilities as defined in the Master Drainage Plan is carried out. During the implementation

phase, extensive environmental damage can result from land development activities (e.g. surface and stream erosion and subsequent sedimentation in downstream waterbodies). Mitigative measures should be implemented during these operations. The scale of these projects relates to residential or industrial subdivisions.

Watershed Drainage Plans and Master Drainage Plans are the key elements in planning for stormwater management. The objectives of these plans are therefore discussed in detail in the following sections.

4.3.2 Watershed Drainage Plans

General

The development of an integrated stormwater management plan requires the definition of an area within which the interactions between individual elements of the drainage system may be significant. Where large scale land use changes will occur within a drainage basin, Watershed Drainage Plans should be developed through comprehensive studies of the existing systems and future drainage requirements. Such studies should be comprised of:

a) a detailed analysis of the existing drainage system and identification of environmental concerns, and
b) an analysis of future drainage requirements and options available for meeting those requirements.

There are no standards to determine the scope of a Watershed Drainage Plan. The downstream area of influence of land use changes may extend beyond the local drainage basin. Logically, the area covered by the plan should extend to where potential impacts are negligible, or where higher level studies or statutory regulations have established limits for the impacts of surface runoff on a receiving stream or water body.

The basic objectives of a Watershed Drainage Plan should be:

a) to provide for an acceptable level of flood protection for existing land uses throughout the basin,
b) to establish constraints within the system to prevent environmental damage, and
c) to develop policies and design criteria for use in the development of comprehensive plans for drainage improvement which may be required in the future.

Watershed Drainage Plans are usually developed at a conceptual level, at least where the drainage basin is largely undeveloped. The basic elements of the Watershed Drainage Plan should include:

a) a statement of objectives,
b) delineation of the watershed(s),
c) delineation of internal sub-basins,
d) identification of the constraints governing the development of the plan,
e) location and capacities of the principal drainage routes,
f) identification of major impacts,
g) formulation of an optimum conceptual drainage plan for the basin,
h) guidelines for implementing the plan, and
i) criteria for designing system components which will meet the objectives.

Basic data required to develop a Watershed Drainage Plan include:

a) a long range land use plan,
b) a biological inventory of the drainage basin,
c) a set of constraints for potential environmental impacts, and
d) methodologies for carrying out hydrologic, hydraulic, and (possibly) water quality analyses.

The format of a Watershed Drainage Plan and the level of effort required to develop it will vary from system to system. In a drainage basin where significant land use changes are not expected, the plan may be a simple document which delineates the plan boundary and sets out an approval procedure for local drainage improvements based on conservative assumptions. In addition, the plan should set out requirements or guidelines for detailed studies if a need arises, perhaps by reference to a standard document.

Watershed Drainage Plan Activities

a) Initial Hydrologic Activities - These include defining basin boundaries, physical characteristics, and estimating the runoff regime for existing conditions. Streamflow records are the ideal source for this information, but are rarely available. Various techniques are available for synthesizing flow data and are discussed in subsequent sections of this report.

The hydraulic capacity of the existing drainage facilities should also be estimated, where applicable. Typically, the system capacity is limited by one or more restrictive elements such as road crossings or channel constrictions imposed by existing land uses. The benefits of eliminating local constrictions should be considered.

b) Flood Levels - Once the watershed hydrology and hydraulic constraints are established, floodplain delineations to determine flood lines for various flow frequencies under existing conditions can be determined. It is common practice, and accepted policy in Alberta, to define flood protection requirements in terms of a flood with a 1 in 100 year return period. Therefore, it would appear to be redundant to estimate flood lines for lesser return periods, although considerable caution should be exercised in this regard. The definition of a design flood line can have a major impact on the overall economics of a project. If the initial flood level estimate is overly conservative, a viable project may be abandoned. Conversely,

if the initial estimate is too low, unexpected costs may be incurred in the future.

The degree of reliability that can be placed on an estimate of the amount of runoff generated by a rare event will vary depending upon the reliability of the base data used to make the estimate. The estimation of higher frequency (common) events is inherently more reliable than for low frequency (rare) events, principally because they can be related to experience and observation. A series of flood lines provides a basis for evaluating both the reliability of an extreme flood projection and the sensitivity of the development costs.

c) Constraint Identification - At this point other sites sensitive to potential adverse impacts should be identified, including sites with erosion or sedimentation potential, and biological communities.

In the quest to preserve the natural environment to the greatest extent possible, it is becoming the norm for authorities to assemble biological inventories of lands within their jurisdictions. Where up to date inventories do not exist they may be required as was the case in recent Watershed Drainage Plans carried out by the City of Edmonton. Such an environmental inventory may include the identification of archeological sites, pedology data, geological data, geotechnical data specifically relating to streamcourse stability, quantification of both upland and wetland habitat, and an estimation of existing life forms in the system.

Potential impacts are not solely attributable to stormwater management practice. If a biological community will not survive the proposed changes in land use, then potential adverse impacts due to drainage modifications are irrelevant. Direct stormwater management impacts are usually limited to the immediate environs

of natural watercourses and water bodies. Specifically, these would include changes in the flow regimes (especially seasonal changes), streamflow velocities, and water quality.

d) Future Flow Conditions - Methods of estimating the magnitude and frequency of future flows occurring in the basin must take into account the effect of increased imperviousness of the land surface, and the effects of improved surface grading and improved channelization. Peak flow rates alone are not indicative of the total change in the hydrologic regime which would occur in the drainage basin. Changes in runoff volume are equally and often more important. As well, natural environments can be more sensitive to changes in average conditions than to the short term effect of occasional high flow conditions.

Once the future hydrologic regime has been estimated, problem areas can be identified and mitigative measures can be devised and evaluated. Mitigative measures fall into two categories: channelization and flow regulation.

Channelization includes deepening, widening and straightening of natural channels or the construction of diversion channels to accommodate upstream inflows. In some cases, floodproofing of specific sites by dyking and pumping may increase the capacity of existing channels.

Flow regulation techniques use temporary impoundment of runoff to reduce flows to rates which can be accommodated in a downstream system.

e) Economic analysis - Economic analysis of stormwater management alternatives is a major consideration in developing a Watershed Drainage Plan. While benefit-cost analysis is the most comprehensive approach to identify the 'best' solution, it is often

difficult to carry out. This is especially the case if there are significant environmental protection objectives for the stormwater management plan. The aesthetic values of retaining natural vegetation or of creating an artificial lake are quite intangible. The benefits of incremental levels of flood protection are also difficult to assess. While recognised methodologies are available to estimate flood damages, they involve a major effort which is generally out of proportion to the overall planning effort.

Probably the most appropriate approach to the economic evaluation of management plans is to determine the cost-effectiveness of alternatives, based on the tangible aspects. This approach requires that intangible environmental aspects be initially evaluated by some empirical process, and those aspects deemed to be important are then included as objectives of the plan. The mitigative measures for the important environmental issues then become a requisite part of the Watershed Drainage Plan.

The cost-effectiveness of an alternative should be evaluated on the basis of total costs over a long term and should include both capital costs and operating and maintenance costs. The major advantage of this "Life-cycle" approach is that effects of staging the implementation of components of the plan in step with the staging of development can be properly evaluated.

4.3.3 Master Drainage Plans

General

The purpose of a Master Drainage Plan is to ensure that the optimal drainage system is developed to meet present and future requirements. The drainage area included would either be determined by existing drainage boundaries (natural or man made) or boundaries imposed by a Watershed Drainage Plan. The Master Drainage Plan is developed through the evaluation of alternatives which provide

an acceptable level of service while meeting the objectives of the Watershed Drainage Plan and satisfying constraints imposed by topography, existing and proposed land uses, land ownership and other local considerations.

The minimum scope of the Master Drainage Plan is to identify and locate major drainage facilities including trunk sewer routes, open channel routes, storage facilities (and their associated floodplain levels and areas), and land requirements for drainage purposes. If sufficient land-use planning information is available, preliminary designs of the major facilities may be developed in the plan.

Master Drainage Plan Activites

The first step is to define the existing local drainage system. Where a natural drainage system is well defined it will invariably become the major drainage system. In such cases it will rarely be economical to introduce radical topographic changes that significantly modify the natural drainage system.

In many parts of Alberta the natural drainage system can be classified as poor. In such cases of poorly drained land, the natural drainage is internal towards potholes, sloughs or marshes. Surface discharges from these collection points are rare or nonexistent. If one of the drainage objectives is to reclaim these flood prone areas, it is necessary to force a major drainage system into the plan. In some cases, the sloughs, marshes, etc. may be integrated into the drainage system.

The second step is to assess the need for discharge controls to meet downstream hydrologic constraints identified in the Watershed Drainage Plan. Increased in-system storage may be required in order to control discharge rates. This can be accomplished by using existing creeks or excavated channels and floodplains, using artificial storage lakes, or by retaining existing sloughs or marshes. Storage facilities may also be beneficial for economic reasons such as reducing the size of long outfall sewers. This can achieve considerable cost savings.

Where in-system storage is required it must be located to intercept overland flows from major events as well as flows from the more common events. Once storage facilities and the major system routes have been located, major trunk sewer routes can be determined. This can be followed by the location and design of the minor system piping.

Computer simulation models have proved to be extremely useful in developing Master Drainage Plans. A number of the more commonly used models are discussed in Section 6.

Land for Stormwater Storage

Provision of land required for stormwater storage lakes may sometimes cause problems in allocating development costs or where development of fragmented land holdings is concerned.

One particular problem in Alberta is that the Planning Act does not provide a zoning category for stormwater management lakes which is equitable to both the municipality and the private developer. One allowable classification is as a Public Utility Lot (PUL). This option will penalize a developer with a storage facility located on his land, as PUL's are dedicated from net area and are subject to all municipal service levies. The only other allowable classification is that of Municipal Reserve (MR). The Planning Act limits MR to 10 percent of the gross developable area. Storage lakes would generally absorb 20 to 30 percent of this area, which in most cases would be quite inequitable from a municipal perspective.

The status of existing wet detention pond sites in Alberta has generally been resolved either through a negotiated agreement between the developer and the municipality, or having ponds constructed on municipal land. Typically, the normal water surface area has been treated as if it were Environmental Reserve, although it cannot be zoned as such. This in effect has reduced the gross developable area from which other dedications and levies are determined.

Problems may arise when a drainage plan identifies a site required for a storage facility. If the land holding is small enough that the facility restricts the developability of the parcel, it devalues the individual property; conversely, a landowner may be in a position to hold a municipality to ransom. Thus, the implications of land requirements should be a specific consideration during the development of a Master Drainage Plan.

4.4 THE DEVELOPMENT OF DESIGN STANDARDS

Many communities have developed design standards which establish the requirements for their infrastructure. These deal with the design and construction of roads, curbs and gutters, sidewalks, water pipes, sanitary sewers, storm sewers, service connections, power, telephone and cable television. Although not all communities have standards which deal with all of these utility systems, most have standards which deal with sanitary and storm sewers. These can range from a concise statement of the physical and design requirements for the sewer systems to the identification of a comprehensive systems planning approach. The latter category covers all activities (from developing Watershed Drainage Plans through to construction), to which a developer and the municipality must conform in order to implement a subdivision. The City of Edmonton (1980) is the only municipal authority in Alberta to date to have developed standards in the comprehensive category.

Although design standards are limited to scope, they are valuable in that they present the philosophy of the municipality with respect to stormwater management. They provide the minimum levels of service and analysis expected, and where more than one option for design is possible, they clarify local preference. Design standards also serve to identify points of local importance which might be overlooked (e.g. the groundwater table, soils characteristics and location of the 100 year floodplain for the local streamcourse).

Design standards have been criticized because they tend to discourage innovation and are unable to address site-specific problems (e.g. soil foundation conditions

and sulphate content). In considering this limitation, Theill (1976) indicated that the following clause should preface all design standards:

> "These standards shall not be considered as a rigid requirement where variation will achieve a better technical and/or economical solution. Indeed it is encouraged that consulting engineers continuously seek new and better solutions."

This flexibility is of value for larger communities that either retain consultants or have staff who can review the technical content of development proposals and design submissions in detail. However, where the commitment to provide this review is not possible, or where the magnitude of the development is small, the utilization of a reasonable set of design standards should be required. With proper application, design based on such standards will provide an adequate level of service.

An example of a typical set of design standards is presented in Appendix A. For many items, acceptable standards can be achieved by requiring one of several options. The items and standards given are intended to reflect what is felt to be the most appropriate or most common practice in Alberta. The Town Engineer is left with considerable authority as to the selection of the details of the Design Standards, as he is responsible for their interpretation (the Town Engineer may be on staff or be a Consultant working on the Town's behalf). As a result, **the Design Standards presented in Appendix A should be reviewed in detail and revised according to local conditions before incorporation into any Bylaws or Development Agreements.** Background information for the various items in Appendix A is given in Section 5 to assist in the development and interpretation of Design Standards.

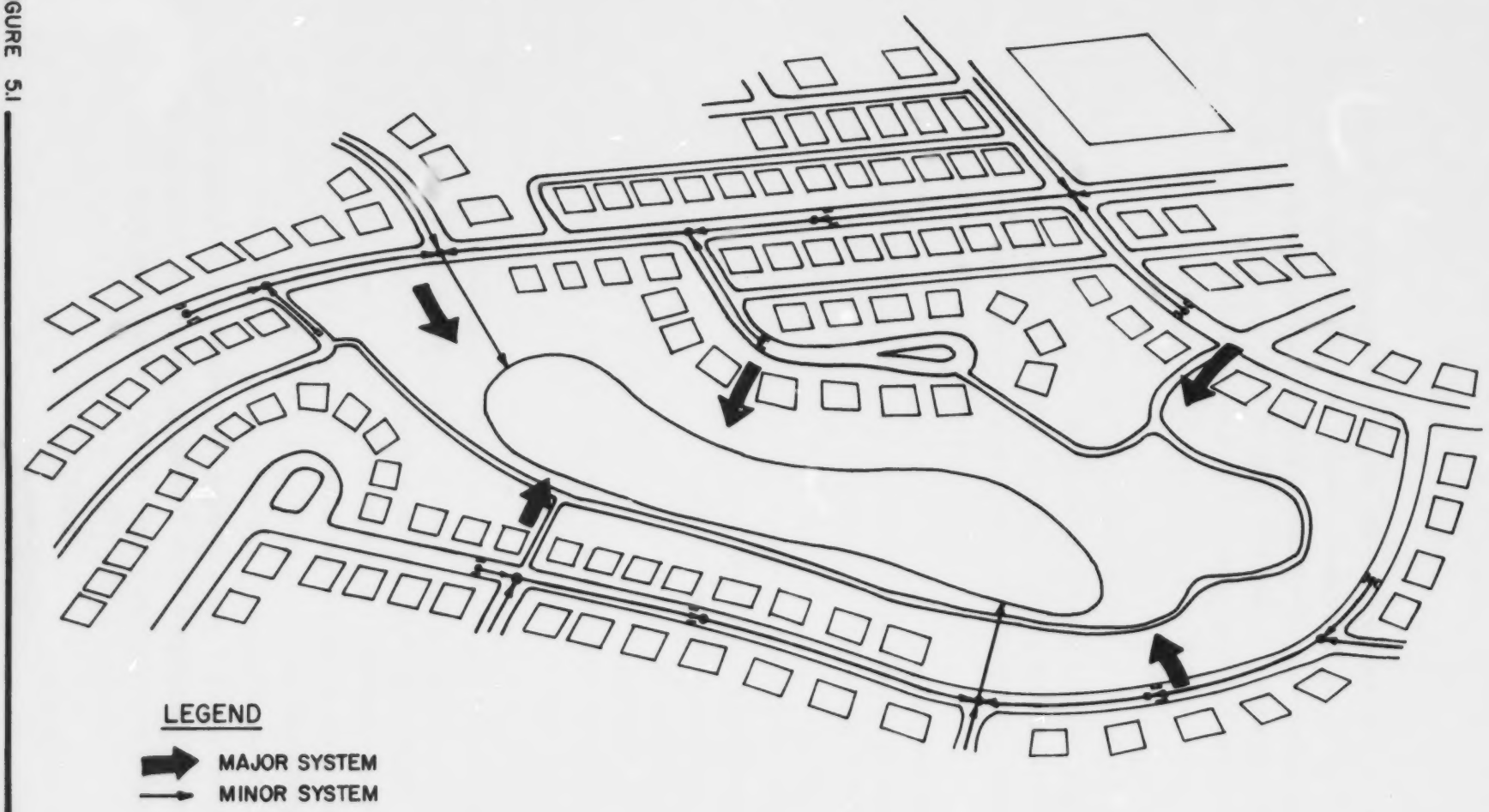
5.0 DRAINAGE SYSTEMS FOR URBAN AREAS

5.1 THE DUAL DRAINAGE CONCEPT

Traditional stormwater drainage systems have consisted primarily of underground pipes and related works designed to transport flows for relatively minor rainstorms. "Minor" in this context means the most severe storm which would be expected to occur, on average, once in a period of (for example) 5 years. Although return periods varying from 2 to 10 years are used as design standards in different municipalities, the most common design criteria for the underground pipe system has been the 5 year return period storm. Historically, there was little or no consideration given to controlling the runoff from larger storms. During major events, meaning larger than the 5 year storm, numerous flooding problems typically occurred.

The solution to the problems which occurred during these infrequent but large events has been to make allowances for major events in the planning and design of new land developments. The division of storms into minor and major events and the realization that separate systems (the minor and major systems) are required for each became known as the "dual drainage" concept (Figure 5.1). The minor system provides a basic level of service by conveying flows from the more common events. The major system conveys runoff from the extreme events in excess of the minor system capacity (providing flood protection usually up a 100 year return period storm). There is always a major system, whether or not one is planned. Failure to plan for a major system often results in unnecessary flood damage. It is now common practice to provide for a major system during the design of any land development. This does not necessarily imply an expensive stormwater management study; it is often only necessary to examine development grading plans to ensure that there is an overland flow path with reasonable capacity.

FIGURE 5.1



MAJOR/MINOR SYSTEMS

Some municipalities (Scarborough, Ontario being an example) require that flow depths and velocities be calculated in the major system. This requires a computational technique capable of separating major and minor system flows and routing them through the catchment. OTTSWMM is an example of a computer model capable of this type of calculation.

5.1.1 The Minor System

The minor system consists of those drainage works which transport flows from minor rainstorms quickly and efficiently from a catchment, providing convenience drainage. The components of the minor system include:

a) roof leaders,
b) foundation drains (occasionally),
c) lot drainage,
d) gutters,
e) catchbasins, inlets and leads,
f) underground pipe systems,
g) manholes, junctions and outfalls,
h) storage facilities,
i) outfall channels,
j) erosion protection and energy dissipators, and
k) receiving waters.

The division of facilities into minor and major system components is not precise; some components, roof leaders for example, carry flow during both minor and major events. Such components are listed as part of both systems since they must be considered in the design of each.

5.1.2 The Major System

The major system consists of those drainage routes which transport flow during major storm events. Ignoring the major drainage system can result in potentially serious flooding of property during major rainfall events. The most common

consequence of a poor major drainage system is property damage by overland flows. Unplanned major system runoff can also create hazards to life as it flows and ponds at locations where residents, particularly young children, are unused to it. Open manholes whose covers have been lifted off by surcharged storm sewers pose another risk, as do roads covered by water. Serious accidents can occur when drivers come suddenly upon flooded underpasses and low points.

The level of analysis and the effort to design facilities to transport major system flows must balance the relatively infrequent occurrence of such events and the seriousness of the damage they cause. It is generally accepted, however, that the level of effort directed towards planning for major drainage events in urban developments has been insufficient in the past.

The components of the major system include:

a) roof leaders,
b) lot drainage,
c) roads and gutters,
d) swales,
e) storage facilities,
f) outfall channels, and
g) receiving waters.

5.2 LEVEL OF SERVICE

The objective of urban drainage systems is to provide a high degree of drainage service without causing unacceptable downstream impacts. This must be accomplished at a reasonable cost, leading to trade-offs between cost and level of service. The level of service provided by a system depends upon the interaction of the various system components. Problems caused by overloading the individual components vary between systems.

In the past, level of service has been established in a relatively simplistic manner, by designing underground pipes to carry the peak flow resulting from a 1 in 5 year rainfall event and spacing catchbasin inlets at suitable distances. The computational methods used to size the pipe systems could result in uneven capacities throughout the system - often the pipes in the upper portion had capacities above the design objective while pipes in the lower portion had capacities below the design objective. Since no consideration was given to the major events there was no consistency in the depths or flows in the major system or the extent to which flooding occurred.

The operation of each component of a drainage system must be examined to establish an even overall level of service at a minimum cost. Modern municipal drainage standards accomplish this through the provision of specific criteria for each drainage system component. Before establishing such criteria it is important to understand how the minor and major systems behave, both at and above their nominal design capacities.

5.2.1 System Behaviour During Minor Storms

The minor system should quickly and efficiently remove the runoff from rainstorms below its design capacity. This runoff comes primarily from the impervious surfaces of a catchment: roofs, driveways, parking lots and roads. Gutter flows are shallow and enter the piped system through catchbasins. The runoff then travels downstream through the pipe system to an outlet where it may be discharged either to a storage facility or directly to a receiving watercourse.

5.2.2 System Behaviour During Major Storms

At some point during a major event the capacity of the minor system is exceeded and overland flow conveyance begins. Depths of flow in gutters will be deep, so at points they will overflow to overland flow routes. These routes are usually swales leading either to storage facilities or directly to receiving waters. The minor system may also surcharge and flow under pressure at some locations.

This should not be a problem unless weeping tiles or roof leaders are connected to the storm sewer system.

For systems with weeping tile or roof leader connections made inside the house, the amount of water entering the pipe system should be limited to what the system can transport without causing damage either to itself, to basements or to downstream receiving waters. If inflows are not properly regulated, sewer systems flowing in a surcharged condition will result in the backup of water with the potential to cause basement flooding or structural damage. Water can enter basements through the connections or cracks or open joints; for well-sealed floors, a pressure head of as little as 0.15 m of water is sufficient to crack the concrete.

5.2.3 System Behaviour Above Capacity

A properly designed dual drainage system should be able to function effectively well beyond its design standard. The open channels and detention facilities which comprise the major system will flow at depths and velocities greater than desirable, but apart from increased erosion in some areas there should be no appreciable damage. If structures are sited well above the design flood level, they should not be flooded except during the most extreme events.

5.3 DESIGN CAPACITIES

As noted in Section 5.2, the selection of nominal design capacities is dependent on the level of service desired, which may vary from one municipality to another. Economic analysis is rarely conducted to determine the optimal design event for drainage facilities; considerations and common practice related to setting design capacity requirements are outlined in the following sections.

5.3.1 Minor System Design Capacity

The establishment of capacity criteria for the minor system is largely a trade-off between cost and convenience in terms of level of service. From a

convenience point of view it is desirable to set the standard on the high side, say the 10 year capacity. This, however, can become quite costly because of the increased pipe and excavation costs and possibly greater costs for downstream flow controls in comparison to a 5 year capacity.

In the larger urban centres in Alberta, the return period for minor system design most commonly used is the one in 5 year event. It has been argued that the 2 year event should be considered. For small communities faced with limited financial resources, the use of the 2 year event may be practical.

5.3.2 Major System Design Capacity

There is more variation in the selection of design standards for the major system because the concept of the major system is comparatively recent. The most commonly used major system design event is the 100 year event. The use of the 100 year event for major system analysis appears to have followed from the de facto standard for riverine flood control projects throughout Canada and the United States. This philosophy is considered prudent.

5.4 DRAINAGE SYSTEM COMPONENTS

The previous sections have discussed the minor and major systems in terms of their overall behaviour during runoff events both at and above their design capacities. This section discusses the individual components of the systems and the considerations relevant to their design.

5.4.1 Roof Leaders

The most common means to accommodate roof drainage in Alberta is to direct it to ground that is graded away from the building. In a few systems, roof leaders are connected directly to the storm sewer. The latter practice can result in very large flows in the minor system and requires that the capacity of the system be increased accordingly. Roof leader connection is discouraged or prohibited in

most jurisdictions. Most favour the discharge of roof leaders onto grassed areas, sometimes by means of "splash pads", well away from house walls. Discharging to pervious areas reduces the volume of runoff through infiltration and retards it through overland flow. Surface discharge of roof leaders is recommended by Alberta Environment.

5.4.2 Foundation Drains

Foundation drains, sometimes called "weepers" or weeping tiles, are the source of many urban drainage problems as they have the potential to cause basement flooding. This can occur when they are connected to either the storm or sanitary sewer systems. For this reason, more care must be taken in the design of foundation drain outlets than has been the case in the past.

On the Prairies, it is common practice to discharge foundation drainage from weeping tile systems to the sanitary sewer. This practice relies on a small flow contribution from weeping tiles due to the low permeability of most soils native to the Prairies. If the sanitary sewer system is a "tight" system (i.e. minimal leakage into the system from poor lot drainage, manhole inflow, cross connections and the illegal or inadvertant connection of roof drainage) then this method works well. In this case, the cost of extending and deepening storm sewers to pick up foundation drainage from all houses in a system cannot be justified.

The practice of connecting foundation drainage to the sanitary sewer system is a problem in cities where poor lot grading exists. As an alternative, foundation drainage can be accommodated by discharge to groundwater through a gravel packed recharge well (although this is not feasible in areas with impervious subsoils) or by use of a sump pump system.

In Alberta, it is recommended that each municipality consider its own situation carefully. If the local soils are generally impermeable and the water table is below most house foundations, then weeping tiles can be connected to the sanitary sewer (a generous allowance for infiltration and inflow should still be allowed for in the sanitary sewer design). If the soils are permeable and the

water table is high then a detailed evaluation of servicing by either connection to the storm sewer (with catch basin inlet controls and/or sewer backup valves), sump pumps, or three pipe systems (see Section 5.4.7) should be carried out.

5.4.3 Sump Pumps

Sump pumps can be used as an alternative to connecting foundation drainage to the sanitary sewer system. They can also be used in existing systems which experience storm or sanitary sewer backup. The foundation drainage flows to a sump in the basement and is removed by the pump. Sump pumps must not be discharged to sanitary sewer systems as the simultaneous discharge of several sump pumps can cause overloading. Storm sewers can accommodate the sump pump discharge, but if roof leaders discharge to the ground there is no need to provide storm sewer service to a house (the usual practice in most Alberta locations). Sump pumps should be required to discharge to the ground, and the ground should be properly graded away from the house.

5.4.4 Lot Drainage

Proper lot drainage is an essential component of good stormwater management. It is important that the grade adjacent to new buildings be sufficient to allow for settlement of the fill and maintenance of positive drainage away from the structure. In many older areas, roof drainage runs out of the leaders only to flow back to the house and down the basement wall. This causes basement wetness and, in severe events, can cause backup of the sanitary sewer in areas where the foundation drains are connected to it. In some drainage systems, rear lot drainage is picked up by catchbasins along the rear lot lines. This is acceptable only under extenuating circumstances (due to high maintenance costs). It is also important to ensure that the ground elevation around building perimeters is well above the levels expected in the major system.

5.4.5 Catchbasins

The spacing and capacity of the catchbasins should be such that the widths and depths of flow in the gutters are acceptable during minor events. The relationship between the amount of water reaching an on-grade catchbasin and the amount which enters the catchbasin is called its "capture ratio" (Wilson, 1983). Proper spacing and capture ratio are important considerations in ensuring that the minor system provides the intended level of service.

Catchbasins are usually constructed with a sump which will trap silt and other settleable debris which can pass through the grating. Some municipalities are more diligent than others when it comes to the regular removal of these accumulations. If a municipal storm sewer system is designed for self scouring velocities, and the municipality is not cleaning the sumps regularly, it is preferable to exclude sumps in the design of new systems.

5.4.6 Pipe Systems and Outfalls

There is an abundance of literature and standards dealing with the design of underground sewer pipe systems. If the amount of water entering such systems is properly regulated, they can be expected to perform as designed. The main difference between traditional and recent design practice is the consideration of inlet controls and the analysis of the behaviour of the system under major events.

The analysis of erosion problems at outfalls is now quite commonplace, with erosion protection and energy dissipators constructed as necessary at outfalls (see Smith, 1978; and Hodgson and Wilson, 1980).

5.4.7 "Three Pipe" Systems

An alternative solution to the problem of foundation drainage is to employ a third piped system which carries only foundation drainage. This provides good and virtually fool-proof drainage to basements and allows the storm sewers to

surcharge with virtually no consequences. In systems where weeping tiles are normally connected to the storm sewer, the three pipe system may allow a 2 year capacity for the storm sewer and may reduce the cost of storm sewer servicing enough to justify the cost of the third pipe. Support for such a system is by no means unanimous although it has been successfully applied in some areas and does provide reliable protection to basements. However, as this system requires another lead to the house, there is the possibility of cross connection between the sanitary sewers and foundation drains.

5.4.8 Roads

During the peak period of a 100 year event, flows will likely fill local roads entirely; flow depths of no more than 0.30 m at the gutter are desirable. Standing water at low points should not exceed 0.50 m or extend to adjacent buildings. For arterial roads, the depths of flow should be less; typical criteria are that two lanes of traffic remain open and that the depth of flow be not greater than 0.05 m where major drainage flows cross arterials. No buildings should be allowed in the area flooded by the major event unless they have been specially designed with floodproofing techniques to withstand flood waters.

5.4.9 Gutters and Swales

Gutters convey flow to the catch basins during minor rainfall events. During these events the depths of flow are usually small and of little consequence. During major events, the much higher flows are conveyed in the gutters and in overflow swales. In these instances, the velocities and depths of flow should be examined more carefully.

Velocities in overland channels should be minimized. The force of moving water on objects in its path increases with the square of velocity. Table 5.1 lists approximate flow depths that a child (20 kg) would be able to withstand while standing in a concrete bottom channel or gutter flowing at the selected velocities. If the public has access to the flow route, these combinations of gutter velocity and flow depth should not be exceeded in open channel design.

TABLE 5.1

PERMISSIBLE DEPTHS FOR SUBMERGED OBJECTS

Water Velocity (m/s)	Permissible Depth (m)
0.5	0.80
1.0	0.32
2.0	0.21
3.0	0.09

Note: Based on a 20 kg child and concrete lined channels.
Larger persons may be able to withstand deeper flows.

5.4.10 Receiving Waters

Consideration of the impacts of stormwater discharges on receiving waters should be implicit in modern stormwater management, with erosion, flooding and water quality being the main concerns as discussed in Section 4.2. It is important to recognize that receiving waters form a part of the drainage system, and that the consideration of drainage does not end at the boundary of the development under design.

5.4.11 "Super-pipes"

Super-pipes (discussed in more detail in Section 5.5.2) are sewers which have been oversized to provide storage for runoff control. They are occasionally used in areas of existing development where trunk sewers are overloaded and the cost of land is high.

5.4.12 Outfall Channels

In many cases urban development does not extend to a receiving water body. In such cases outfall channels or ditches are the most economical means of

conveying stormwater to the receiving water body. The use of open channels within the urban developments has generally been avoided due to safety, poor aesthetics, or high maintenance costs. Higher land values also reduce the economic advantages of channels within urbanized lands. There are some potential uses for open channels in stormwater management for urban areas in association with dry ponds as discussed in the next section.

5.5 RUNOFF CONTROL

The need for runoff control has been discussed in Section 4.2. To recap, urban land developments greatly increase the volume and rate of runoff, mainly as a result of the large areas of impervious surfaces which they contain in the form of roofs, driveways, parking lots, and roads. The increased rate of runoff can usually be controlled by means of stormwater storage facilities which temporarily store the excess runoff and release it at a controlled rate. Normally, little can be done about the increased volume of runoff except in those few areas where infiltration or evaporation facilities are feasible.

The forms of runoff control include:

a) wet ponds,
b) dry ponds,
c) parking lot storage,
d) rooftop storage,
e) super-pipes,
f) catchbasin inlet controls,
g) infiltration areas,
h) soak-away pits,
i) cisterns,
j) evaporation areas/ponds,
k) splash pads to pervious areas, and
l) regulations limiting impervious areas.

5.5.1 Detention Ponds

As detention ponds are the most commonly used form of runoff control, a discussion of design considerations for these facilities is relevant. Detention ponds are also further identified as being either wet or dry ponds. Wet ponds are popular in many areas largely because they provide an aesthetic and recreational amenity.

Design Objectives and Effectiveness

Most ponds are designed to control runoff for a range of storms in order to meet erosion or flood control objectives. More recently, ponds have been considered for water quality improvement. Identifying the objective of the facility is the most important and sometimes most neglected stage of the design process. Establishing objectives normally means a review of the characteristics of the receiving water. This is an undertaking which is best performed by government on a Watershed Drainage Plan level of analysis, as discussed in Section 4.3.2.

A common objective for a stormwater pond has been the matching of post-development to pre-development peak flows. This simple matching of pre and post-development peak flows is not always an appropriate objective. Even if peaks are properly matched, the duration of flow under post-development conditions will be many times longer than for pre-development conditions. Events which previously caused only minor erosion as the peak quickly passed through the system in the pre-development condition, can cause extensive erosion damage after development. Also, the season of peak discharge may be changed. In many watersheds the maximum annual flows are caused by snowmelt. Urban development in part of the watershed may result in several periods of high flow during a year, one in the spring during the snowmelt and several others as summer storms occur over the urban areas. This can cause serious problems for farmers using low-lying land adjacent to watercourses.

In large urbanizing areas the effects of many ponds on downstream flood peaks also needs to be considered at a Watershed Drainage Plan scale. Normally, peak

flows from different areas of a watershed do not coincide at a downstream point due to different travel times and variations in the rainfall distribution over a larger watershed. With a large number of storage facilities on a watershed discharging their peak flows for long durations, the peaks will become concurrent and directly additive. This has the potential for large increases in downstream flow.

Wet Ponds

Wet ponds typically have depths of no less than 2 m to maintain reasonable water quality. In some municipalities, minimum water surface areas of about 2 ha are required to prevent a proliferation of small ponds which would result in a heavier operation and maintenance burden. Maximum water levels are sometimes stipulated to limit the levels to which ponds can rise during major events; typical allowable rises are in the order of 2.0 m during a 100 year event.

Outlet facilities for ponds can consist of a concrete weir, a berm with culverts at several levels, a mid-pond draw-off, or any one of a variety of other outlet structures. Side-slopes are typically grassed with rip-rap or gabion erosion protection for ponds large enough to experience erosive wave action. Various edge treatments are also used to minimize on-shore weed growth and maintain aesthetics. The pond side-slopes are normally kept flat in order to reduce the risk of slipping on wet grass and falling into the water. There is a trade off involved here, however, as overly flat side-slopes require more extensive edge treatment to accommodate movement of the water's edge during small rain events or extended dry periods.

Inlet pipes may enter a pond at different levels. In some areas the prime concerns are the avoidance of the effects of submergence on pipe hydraulics and siltation, while others feel that the aesthetics of the pond are improved if pipes have their inverts below the normal water level. For Alberta it is recommended that pond inlet pipes be submerged with their crowns below the anticipated ice level. Large diameter inlet pipes may need special considerations (e.g. multiple pipe inlets). A manhole should be located as near to this inlet as possible, but at

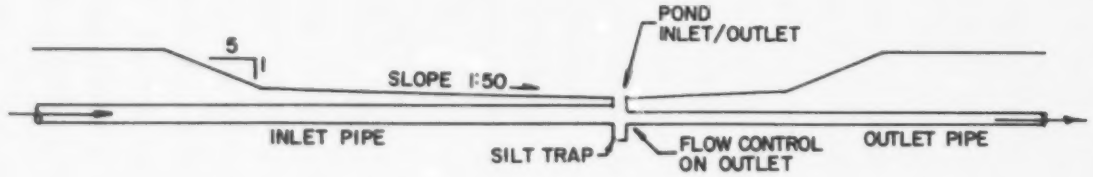
a location where the ground elevation is above the design high water level. Only the inlet pipe should be submerged; all other pipes upstream should flow without backwater effects during the minor system design events (Figure 5.2).

Dry Ponds

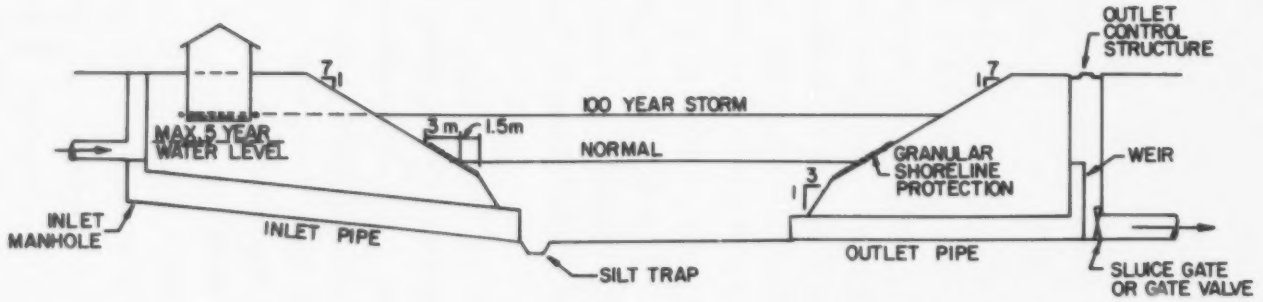
Dry ponds which only contain water during and for a short time after runoff events are gaining in popularity. While many of the design considerations for dry ponds are similar to those for wet ponds there are important differences.

The primary reason for using dry ponds is to maximize land utilization through dual usage of land dedicated for recreational uses. Large grassed areas such as football fields and ball diamonds can provide large storage capacities at relatively shallow depths. In some cases, municipalities have employed dry ponds to avoid potential water quality and maintenance problems associated with wet ponds. However, post runoff event clean up and time lost for recreational use should be considered in evaluating the suitability of dry ponds. In addition to removal of silt and debris, restoration of minor damage to landscaping and facilities may be required. Thus, dry ponds are often not suitable if normal rainfall events would cause frequent inundation.

Siting requirements for dry ponds tend to be less restrictive than those for wet ponds. In particular, the aesthetic impacts of size and shape are rarely significant. In some cases it may be possible to retrofit dry ponds into open space areas in existing developments. The dry pond storage concept can also be applied in a very linear form, such as a floodplain, or a natural or manmade channel used for stormwater drainage. By designing hydraulic constrictions into the channel, discharge rates can be regulated forcing temporary storage during major runoff events. This approach is the basis of the blue-green concept of integrating stormwater facilities with linear parks.



DRY STORMWATER DETENTION POND



WET STORMWATER DETENTION POND

TYPICAL POND CONFIGURATIONS

FIGURE 5.2

Since dry ponds are designed to be dry and useful for active or passive recreation, adequate drainage is essential. Longitudinal slopes will often be as flat as possible to maximize storage in the pond without resulting in excessive depths at the outlet. Bottom-slopes should be at least 1 percent to facilitate lateral drainage. French drains or shallow drainage pipes can be used to prevent flatter sections from remaining wet for prolonged periods of time.

Provision should also be made for a low flow channel, or a bypass conduit through a dry pond in order to restrict the area wetted by very small events (Figure 5.2). Such a pond is an off-line facility which is only inundated when the sewer system capacity is exceeded. Low flow bypass channels for dry ponds will serve to minimize wetness and sedimentation problems. Sediment build-up in flow-through dry ponds can be an eyesore and a maintenance problem, since the clean-out must normally be done by manual labour.

In all areas of the pond, slopes should be flat enough to facilitate grass cutting and reduce the risk of slipping into the pond when it contains water. It is particulary important to have adequate signage in dry ponds warning against the dangers of sudden flooding. Painted markers at several locations showing the maximum water levels will give some idea of the possible extent of flooding.

Safety

The use of stormwater management facilities poses new hazards in the urban environment. As with any relatively new concept or product, it can be difficult to identify what constitutes an acceptable level of risk and good engineering practice. This is changing as stormwater facilities become more common features of urban development.

Each component of the drainage system poses its own risks. Children are naturally attracted to water bodies of all types; this must be considered in the design of stormwater management facilities. Particular care must be taken to avoid the creation of hidden hazards. A number of safety related considerations are:

a) Ponds present an obvious hazard to non-swimmers and skaters. Signs should be posted which clearly indicate the nature of the hazards and which prohibit the inappropriate activities.

b) Outfall structures and energy dissipators also present serious risks and may require fencing to restrict access. All exposed outfalls should be grated to prevent entry.

c) Pond side slopes should be minimized to prevent individuals from slipping in; maximum side-slopes of 7:1 up to the 100 year level are recommended in some design standards.

d) Fencing of stormwater facilities must be considered carefully in each case. Children are expert at defeating fences which may only serve to hamper rescue efforts. Not placing fences may, however, be considered an invitation for children to use a facility and pose legal problems for the owner.

e) Dry ponds pose an unusual hazard in that they are normally dry areas which may rapidly fill with water during extreme events. Children used to playing in such areas may not be aware of the hazard they present when full. Dry ponds should be attended by Drainage Department staff during the period which they contain water.

Climatic Considerations

Alberta's climate is continental and comparatively dry. Winters are severe, and in many areas of the province are unique. This is particularly true in the south which experiences many freeze-thaw cycles as a result of Chinook winds. Summers are characterized in most areas by large variations in temperature throughout the day and high evaporation as a result of winds, low humidity and long hours of sunshine. In the southeast part of the province, annual precipitation is very low and the precipitation deficiency is the greatest. Low precipitation and high evaporation may result in a need for make-up water to

maintain normal pond operating levels and/or acceptable water quality in wet ponds. This need must be carefully evaluated, as it is not always desirable to use groundwater as a source of makeup water. Groundwater is generally considered a valuable resource in Alberta and makeup water is not considered a high priority use. Proper side slope treatment can usually overcome the need for using make up water to maintain a normal water level if an aesthetic appearance can be maintained even when the waters surface is drawn down by 0.3 m.

Snow and ice may cause problems at pond outlet structures, and consideration must be given to blockage of inlet and outlet works. The orientation of the structure to the sun may be important in reducing ice buildup. Snow and ice may also block channels and cause water to flood adjoining areas more frequently than intended. In urban areas, impervious surfaces tend to warm faster than pervious areas, so snowmelt may run off the development area to a pond or channel which is filled with snow. Unique climatic factors must be considered in each design; consideration should be given to the operational history of any similar works built in the area.

Sedimentation

Sedimentation must be considered in the design of both wet and dry ponds. Wet ponds must be capable of being drawn down to facilitate sediment removal or have facilities to pump down the water level. The means of sediment removal should be identified when the pond is constructed. Little data is available for predicting the required frequency of sediment removal; this can vary widely depending on the type of land use in the catchment and whether or not construction is still occurring in the area. Ponds are sometimes separated into two cells, with a smaller upper cell which serves to trap most of the sediment. The cells are separated by a submerged berm. The upper area may incorporate access for sediment removal equipment and can have a concrete surface to facilitate scraping or dredging to a fixed level.

Sediment buildup in dry ponds may be more difficult to control because the bottom of the pond will be grassed, and even a compartively small amount of

sediment may be unacceptable aesthetically considering the recreational uses of the pond. One solution is to intercept at least the coarser sediment before it reaches the pond by trapping it in large "clean-out" catchbasins or larger sediment traps at the inlets to the pond. Another solution is to use the by-pass concept in the pond design. Here, sediment deposits on the grass will occur only after major events (Figure 5.2).

Recreation and Aesthetics

Properly designed stormwater management facilities can provide recreational opportunities and enhance community aesthetics. While the water quality of urban ponds is generally not suitable for body contact recreation, ice-skating may be possible in areas of the province where adequate ice thickness can be maintained for a sufficiently long period of time and the water level can be kept constant to prevent ice breakup from uplift. This type of use places a responsibility and an associated legal liability on the local authority for supervision and maintenance, together with the duty to ensure safe conditions (see Section 8).

The greatest secondary benefit from urban ponds may be their aesthetic appeal. Properly landscaped ponds can provide a very attractive setting for a residential community. Once the engineering considerations have been established, landscape architects should work with the engineers to improve the shape and layout of the pond while maintaining its functional characteristics.

5.5.2 Other Runoff Control Alternatives

There are a number of other runoff control alternatives available for stormwater management. These are discussed in the following, although not all methods will be applicable to Alberta.

Parking Lot and Roof-Top Storage

Commercial and industrial developments normally contain parking lots, storage areas and roof-tops which have potential for stormwater storage. A problem with such facilities is that they are normally privately owned. The local authority may not be able to control the illicit removal of flow control devices which are necessary to provide the ponding in such areas, particularly on roof-tops. In land developments where lots are sold to third parties, enforcing the implementation of the restricted outflow controls and storage on private property is difficult. In commercial areas there may be resistance to ponding on parking lots because of the fear of inconveniencing shoppers caught away from their cars during a rainstorm. For these reasons some municipalities will not accept privately owned storage facilities for stormwater management control.

Super-pipes

Super-pipes are over-sized pipes used to provide underground storage in the minor system. They are normally used only where ponds are infeasible since the cost of providing underground storage is very high. Such locations would be the upstream ends of systems with high density development where open space is limited and land values are higher. Sedimentation can be a serious problem in super-pipes unless they contain low-flow channels, which add further expense. Additional costs should be included in operation and maintenance budgets to ensure the storage facility will be operational when needed. Super-pipes should also contain overflow sections to prevent upstream flooding should their outlets become blocked.

Cisterns, Soak-away Pits and Infiltration Ponds

Cisterns are small covered tanks for storing water for a home or a farm; these are usually placed underground. Cisterns have the potential to provide a high degree of on-site stormwater storage, but their application is limited, probably because of lack of owner acceptability and municipal control. They may present a solution in areas with limited room for ponds and restricted drainage outlets.

Interception of roof drainage into cisterns can greatly reduce peak flow rates to the storm sewer. Cisterns can be designed to drain slowly to the storm sewer, or to soak-away pits (where soil conditions are suitable).

Few areas in Alberta have soils with sufficient permeability to facilitate the use of soak-away pits. Infiltration ponds are also limited in application in urban areas due to the need for large areas with very permeable soils. On the scale of the individual lot, a cistern could drain to a pervious area such as a lawn, but only if the underlying soils are sufficiently permeable.

Evaporation Ponds

Evaporation ponds also require large areas to be functional. Evaporation and infiltration ponds may be viable in rural or semi-rural areas where land is available and soil conditions are suitable.

Limits on Imperviousness

Almost all runoff from minor storms and a large portion of the runoff from major storms comes from impervious surfaces. Limiting the impervious area will directly reduce the amount of runoff generated. However, since the degree of imperviousness is pre-determined by the type of development most economically suitable for a site, it is generally impractical to prescribe limits on the portion of a site which may be covered with impervious surface.

Porous pavements have also been proposed as a means of reducing runoff, but this type of runoff control measure has not been used in Alberta.

6.0 METHODS FOR QUANTITY ANALYSIS

6.1 GENERAL

Stormwater runoff involves the interaction of a number of phenomena. A rigorous analysis of the runoff resulting from a given rainfall event involves a large number of complex calculations. Prior to the general availability of computers, the time and manpower effort required to carry out such calculations could rarely be justified. Thus, the use of simplified or approximate methods based on empirical relationships were commonplace and are still firmly entrenched in urban hydrology.

Many of the problems that have occurred in existing stormwater drainage systems have to some extent resulted from the use (or, more correctly, the misuse) of empirical methods. However, there is no guarantee that more sophisticated methods will eliminate future problems. They simply provide the ability to investigate the cause and effect relationships both in greater detail and with less effort. One important advantage of stormwater management analysis by computer models is that it provides a common basis of assessment for both the developer and the local authority.

Considering the complexity of the runoff process, any method of estimating runoff rates and runoff volumes should be applied with considerable caution. Such analysis requires both an understanding of the runoff process and the way a particular methodology portrays the process. This section provides some background for those involved in stormwater management analysis. The rainfall/runoff phenomenon is discussed, followed by a discussion of various methods for estimating runoff hydrographs and routing them through drainage systems. The section concludes with a description of some of the more commonly used computer models that are available in the public domain.

6.2 THE RAINFALL/RUNOFF PROCESS

The amount and timing of runoff from a watershed is a function of several phenomena which have varying degrees of importance depending on the nature of the system being modelled. The analysis of runoff processes includes the assessment of the precipitation event, interception and depression storage, evaporation and infiltration. These latter items are called losses.

Interception storage is the amount of precipitation that can be stored by surface tension as it adheres to the vegetation in the watershed. This water is later evaporated back into the atmosphere. This may be 1 to 2 mm in forested areas and up to 4 mm in cropped land (Linsley et al, 1975). It is a factor in the annual water budget and can be considered in rural runoff simulation. Interception storage, however, is not a factor during intense, short duration rainfall events that are usually considered in urban runoff modelling.

Depression storage is water retained in puddles, ditches and other depressions in the ground surface. This water may later evaporate to the atmosphere or infiltrate into the ground. For rural watersheds this factor is of considerable importance but cannot be quantified based on land use or land form characteristics. As the nature of surfaces in an urban area is more regular and controlled, depression depths can be estimated. These are typically from 1 to 5 mm on paved surfaces and about 5 to 10 mm on grassed surfaces.

Evaporation and evapotranspiration of water to the atmosphere accounts for a considerable portion of the annual losses from surface water systems; however, it is of little significance in the analysis of peak runoff events in rural or urban watersheds, as evaporation rates are extremely low compared to peak precipitation rates. These water loss mechanisms are, however, factors in the post event soil moisture depletion and the estimation of antecedent moisture conditions for subsequent events.

The infiltration of water through the soil surface is a significant factor in rural and urban watersheds. The soil infiltration rate is typically considered in terms of Horton's relationship:

$$F = F_C + (F_O - F_C)\, e^{-Kt}, \qquad (1)$$

where: F is the infiltration capacity at time t,
Fo is the initial (dry) infiltration capacity,
Fe is the equilibrium (saturated) infiltration capacity,
t is the time since initial infiltration rate Fo, and
K is a the decay rate for infiltration.

The initial infiltration rate is significantly greater than the equilibrium capacity. Many rural and urban runoff models are based on this concept. Although urbanization greatly decreases the area of land available to infiltrate water, infiltration is still a significant loss component in an urban area. Very little of the pervious area contributes to the runoff during normal events, whereas for rare events the pervious area generates a significant amount of runoff. Equilibrium infiltration rates indicated in the literature are in the order of 10 to 20 mm/hr.

6.3 RAINFALL CONSIDERATIONS

The precipitation input for the generation of runoff from a watershed comprises either snowmelt, rainfall or both. Snowmelt can be an important influence on the runoff from rural watersheds. Snowmelt runoff is often dealt with as part of a statistical hydrologic analysis of stream gauging records. Although snowmelt/runoff simulation models are available and are used for large watersheds, they are not often used in assessing runoff conditions in smaller rural or urban watersheds.

Despite being a northern country with the implied abundance of snow and cold weather, the critical runoff events for the majority of Alberta's (and Canada's) cities are rainfall related. For urban areas, rainfall is the single most influential

component in the generation of runoff. Because of the importance of rainfall, rainfall events are the subject of further discussion in the following sections.

6.3.1 Antecedent Moisture Conditions

The Antecedent Moisture Condition (AMC) is a measure of the soil's current moisture content. It can be quantified by analysing the amount of rain that has fallen in the hours, days or weeks prior to a storm. Runoff coefficients, infiltration parameters and other runoff model parameters can be adjusted by knowledge of the AMC. One application of this concept has been presented in the Soil Conservation Service's (SCS) National Engineering Handbook (USDA, 1972). It is used as a means to adjust the Curve Number (an SCS runoff parameter) for a given soil type based on the amount of rainfall which has occurred in the previous five days. An increase in the AMC means there is an increase in runoff potential from a watershed.

The SCS Handbook indicates that the AMC index is only a rough approximation of runoff potential as it does not include the effects of evapotranspiration and infiltration on watershed wetness. In estimating the AMC, their use of the prior 5 day precipitation does not address the greater importance of the rainfall immediately prior to the rainfall event or the effects of a large rainfall occurring prior to the 5 day period. A more refined antecedent precipitation assessment has been in practice for some time where soil moisture is assumed to decrease logarithimically with time during periods with no precipitation (Linsley et al, 1975). The Antecedent Precipitation Index, determined in this manner, is given by the equation:

$$API = \sum_{i=1}^{\infty} I_i K^i \qquad (2)$$

where: I_i is the precipitation on the i^{th} day prior the the rainfall event, and
K is a recession constant (typically between 0.85 and 0.98).

There is little information available on relationships between antecedent moisture conditions and runoff model parameters. In the application of the SCS method, common practice is to use the most probable AMC condition. In Alberta this is typically AMC I (Table 6.1). For urban runoff models, no relationships are presented in the literature for pervious area infiltration rates as a function of AMC.

6.3.2 Intensity Duration Frequency Curves

The amount (depth) of rainfall and the rate (intensity) at which it falls are the most important aspects of a rainstorm. This information is well documented in Intensity-Duration-Frequency curves (IDF curves) which relate the intensity of the rainfall to the duration of occurrence for various probabilities (Figure 6.1). These relationships have been based on short duration rainfall data collected by Atmospheric Environment Services (AES) of Environment Canada from locations which they monitor throughout Canada. There are 28 locations in Alberta for which IDF curves have been developed (Table 6.2). Of these, eight are based on data from 20 or more years of record.

IDF curves are often expressed in functional form. Functional representation facilitates the precipitation data input to design storm generation and other stormwater management computer programs. The form of the most commonly used relationship for an intensity-duration relationship is:

$$i = A/(t + C)^B \qquad (3)$$

where: i is the rainfall intensity,
t is the duration of the rainfall event, and
A, B and C are regression constants.

AES has conducted statistical analysis for the monitoring stations in Alberta. They related the rainfall intensity to the duration (hours) using a simpler geometric regression analyses: $i = A/(t^B)$. This equation results in regressions that have poor standard error of estimate and should not be used. The tabulated

TABLE 6.1
ANTECEDENT MOISTURE DATA - ALBERTA

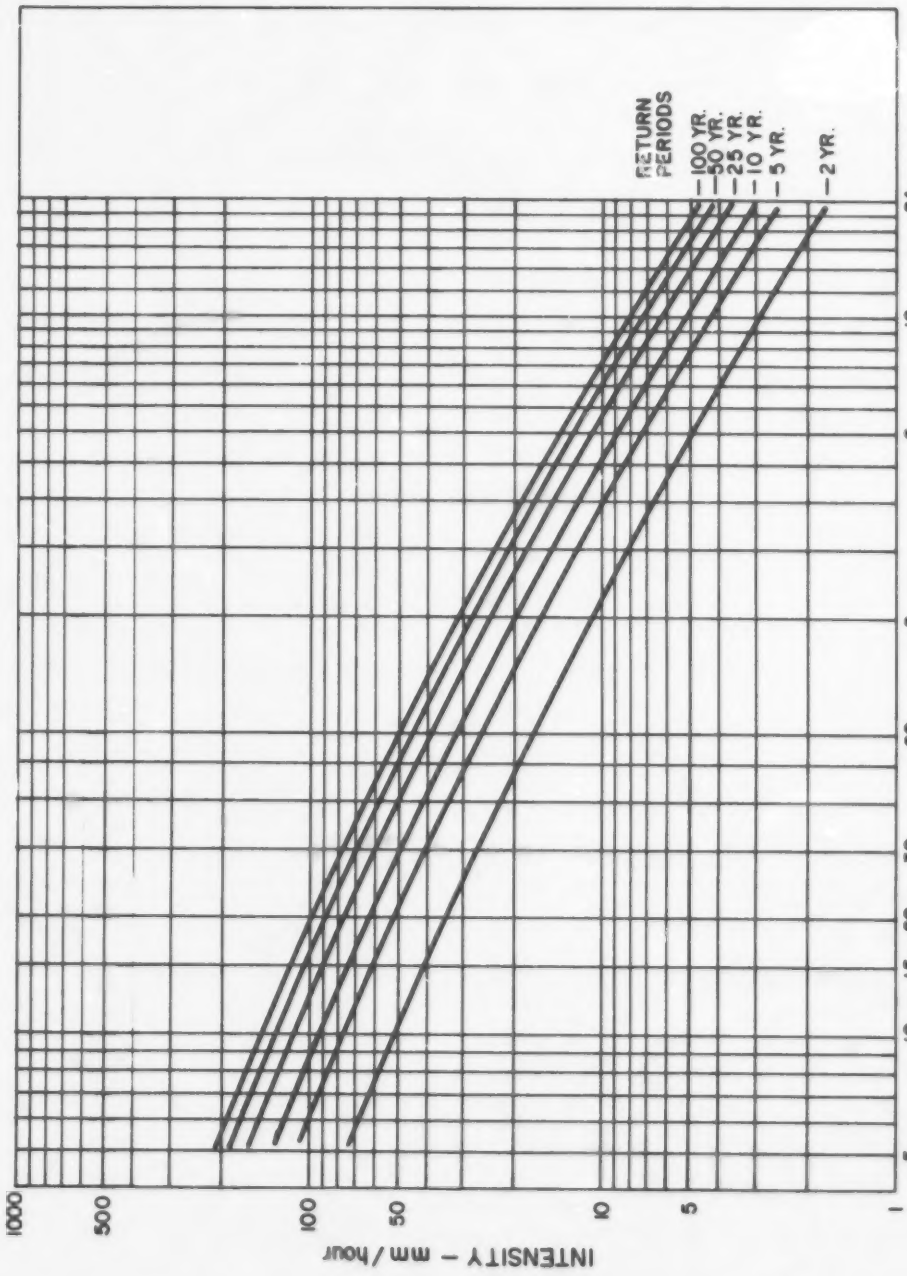
Location	Period	Threshold (mm)	# Events	Antecedent Rainfall (mm) -1	-2	-3	-4	-5	Total (mm)
One Hour Events:									
Edmonton	46-74	15.2	14	1.0	4.2	9.2	2.4	1.8	18.6
Calgary	50-74	15.2	11	3.4	4.2	0.1	2.0	1.3	11.0
Lethbridge	63-74	12.7	9	2.1	0.6	2.3	3.8	0.1	8.9
Vauxhall	58-74	12.7	10	3.9	0.2	0.5	0.3	0.7	5.6
Average	-	-	-	2.6	2.3	3.0	2.1	1.0	11.0
12 Hour Events:									
Edmonton	14-74	30.5	32	8.1	1.5	0.9	0.8	1.8	13.1
Calgary	51-76	30.5	17	6.1	0.6	1.0	1.4	3.4	12.5
Lethbridge	61-76	30.5	18	10.5	2.0	3.5	3.0	1.5	20.5
Vauxhall	56-77	30.5	15	6.1	0.5	1.3	1.1	2.4	11.4
Average	-	-	-	7.7	1.2	1.7	1.6	2.3	14.4

Notes:

1. Based on data from Atmospheric Environment Services, Environment Canada.

2. Soil Conservation Service 5 day AMC categories:

AMC	Dormant Season	Growing Season
I	< 13 mm	< 36 mm
II	13 to 28 mm	36 to 53 mm
III	> 28 mm	> 53 mm



TYPICAL I.D.F. CURVE FORMAT

FIGURE 6.1

TABLE 6.2

ATMOSPHERIC ENVIRONMENT SERVICES RAIN GAUGE STATIONS

RAINFALL INTENSITY - DURATION - FREQUENCY DATA FOR ALBERTA

Station Name	Years of Record	Last Year of Record	5 Year Curve (mm/hr) 10 min	60 min	12 hr	24 hr
Beaverlodge CDA	22	1983	73.1	23.3	3.29	2.40
Brooks AHRC	17	1983	56.6	22.5	3.31	2.01
Calgary A	33	1983	71.6	20.0	3.22	1.96
Cold Lake A	18	1983	79.1	21.7	3.01	1.91
Edmonton Municipal A	43	1974	79.5	25.5	4.28	2.55
Edmonton International	22	1983	70.3	20.3	3.79	2.36
Edmonton Namao A	19	1983	68.3	20.4	3.96	2.61
Edson A	14	1983	86.0	24.0	3.71	2.41
Ellerslie	16	1983	66.5	23.1	4.19	2.65
Forestburg Plant Site	10	1982	96.6	26.6	3.28	2.18
Fort Chipewyan A	15	1983	43.8	14.5	3.00	1.90
Fort McMurray A	18	1983	59.4	17.9	3.86	2.31
Fort Vermillion	7	1972	53.3	20.9	3.45	1.78
Grande Prairie A	15	1983	51.1	14.0	3.27	2.29
Jasper	20	1983	37.7	9.8	2.95	2.03
Lacombe CDA	14	1983	80.2	24.9	4.12	2.63
Lethbridge A	24	1983	93.5	28.7	4.30	2.60
Manyberries CDA	14	1983	52.5	15.0	2.73	1.77
Medicine Hat A	13	1983	76.4	18.6	3.08	2.13
Mildred Lake	10	1983	48.3	16.9	3.72	2.25
Peace River A	8	1983	51.0	18.1	2.78	1.69
Pincher Creek A	19	1983	54.0	17.7	4.27	2.63
Red Deer A	20	1983	65.6	18.3	4.01	2.65
Rocky Mtn House	18	1983	80.4	24.6	4.01	2.46
Slave Lake A	11	1983	53.9	16.8	3.92	2.50
Vauxhall CDA	28	1983	59.8	20.0	3.23	1.87
Vegreville CDA	10	1983	--	23.4	3.30	2.08
Watino	12	1983	64.7	22.4	3.54	2.43

Note: Data can be interpolated if plotted on log-log paper. For other return periods refer to AES.

data from AES should be used. Some representative values for the 5 year IDF curves are presented on Table 6.1. For detailed studies and other return periods the more detailed information should be used (contact AES in Edmonton at 420-3143).

Within any given year, many independent rainfall events occur. This is not addressed in IDF curve derivations by AES, which are based on single annual event statistics. With the AES method, the largest rainfall intensity for a specific duration for each year in the period of record is analysed using the arithmetic Gumbel distribution. Where two or more severe rainfall events occur in the same year, the lesser events will be excluded from the analysis, even though they may exceed events in other years. Fortunately, the effect of this is not significant for the events rarer than the five year return period frequency. Haan (1977) indicated that the intensity derived for a 5 year event in the traditional fashion (single annual event statistics) is really only about a 4.5 year return period event (a correction factor of about 1.04 is all that is required to compensate for this). As a result, there is only minor error in using single annual event statistics in normal stormwater management design.

Some authors propose the two year event for pipe design in dual drainage system analysis. When using the two year event, consideration should be given to the impact that partial duration series analyses would have on the 2 year IDF curve. The two year event estimated in the conventional manner underestimates the rainfall intensities by about 14% (a correction factor of 1.16 would compensate). This has the potential to cause a designer to underestimate the magnitude of 2 year return rainfall intensities.

6.3.3 Spatial Distribution of Rainstorms

The variation of rainfall over a large area has been recognized in watershed hydrology. It has been difficult to quantify in smaller urban drainage areas. It can be significant in the Prairies, where thundershowers may move rapidly and may be quite local in nature. The cities of Edmonton and Calgary collect rainfall data at several sites to form a basis for evaluating spatial effects. As

most urban watersheds are relatively small, it is presently acceptable to assume that a uniform rainfall distribution based on point recordings at a rain gauge will represent average conditions in the watershed.

Using rainfall data from a low level station to represent a high level watershed will underestimate conditions. This phenomenon must be addressed in watersheds where orographic effects are of significance.

6.3.4 Temporal Distribution of Rainstorms

The temporal distribution of rainfall is the variation, with time, of the rainfall intensity during a storm event. A uniform intensity rainfall event is a necessary simplifying assumption for the application of the Rational Method. The data to determine this intensity is conveniently given in the form of the IDF curves discussed earlier.

In small urban drainage basins, the peak flow rates are very sensitive to the storm distribution. For proper hydrograph analysis of stormwater management systems a more realistic temporal distribution of rainfall is needed. Where system storage is significant (either in large drainage systems or those with detention facilities) the design storm configuration is not quite as important.

To provide a reasonable and consistent basis for analysis for urban runoff modelling, a 5 minute time step is recommended for small urban areas. This is commensurate with the resolution of AES data. It is also less than the time of concentration (tc) of the smallest sub-basin that would usually be under investigation in an urban runoff modelling exercise. Typically, one should select a time step about one half of the minimum tc value.

Design storms for a particular frequency event can be developed based on synthetic methods or based on historical data. These are described in the following section.

Synthetic Design Storms (Hyetographs)

The most commonly used synthetic method for developing design storms has been the Chicago method (Keifer and Chu, 1957; Bandyopadhyay, 1972). This method distributes the rainfall indicated by an intensity-duration curve of a selected frequency. The Chicago Method was developed ignoring the likelihood of the short duration and long duration rainfall intensities being concurrent (i.e. the high intensity short duration rains tend to be isolated cloudburst events). As a result, the procedure produces design storms that are too peaky. This has been supported by several investigations. It was found that the peak 5 minute rainfall intensities determined in the synthetic hyetographs are much higher than observations of the peak intensity during recorded 1 hour storms (McLaren, 1979; Wisner and Gupta, 1979).

This overestimation of the short duration intensities is one of the reasons for considerably higher peak flow rates being determined by most computer simulation models. This aspect of the storm hyetograph (the ratio of the peak 5-minute intensity to the average intensity for the storm) was termed the "Peakiness Factor" (or PF value) by Wisner and Gupta (1979). From a review of Toronto Airport Rainfall data they found that the observed PF values (which ranged from 2.2 to 2.8) were much lower than that for the 5 year, 1 hour Chicago type synthetic hyetograph (which had a PF value of 5.9). For a given storm, increasing the time step decreases the PF value and, hence, the peak flow rate. Wisner and Gupta (1979) found that a time step of 10 minutes used in developing the 5 year, 1 hour Chicago hyetograph resulted in a PF of 2.9 (similar to the historic PF values).

The Chicago method for design storm development found popularity for a number of reasons:

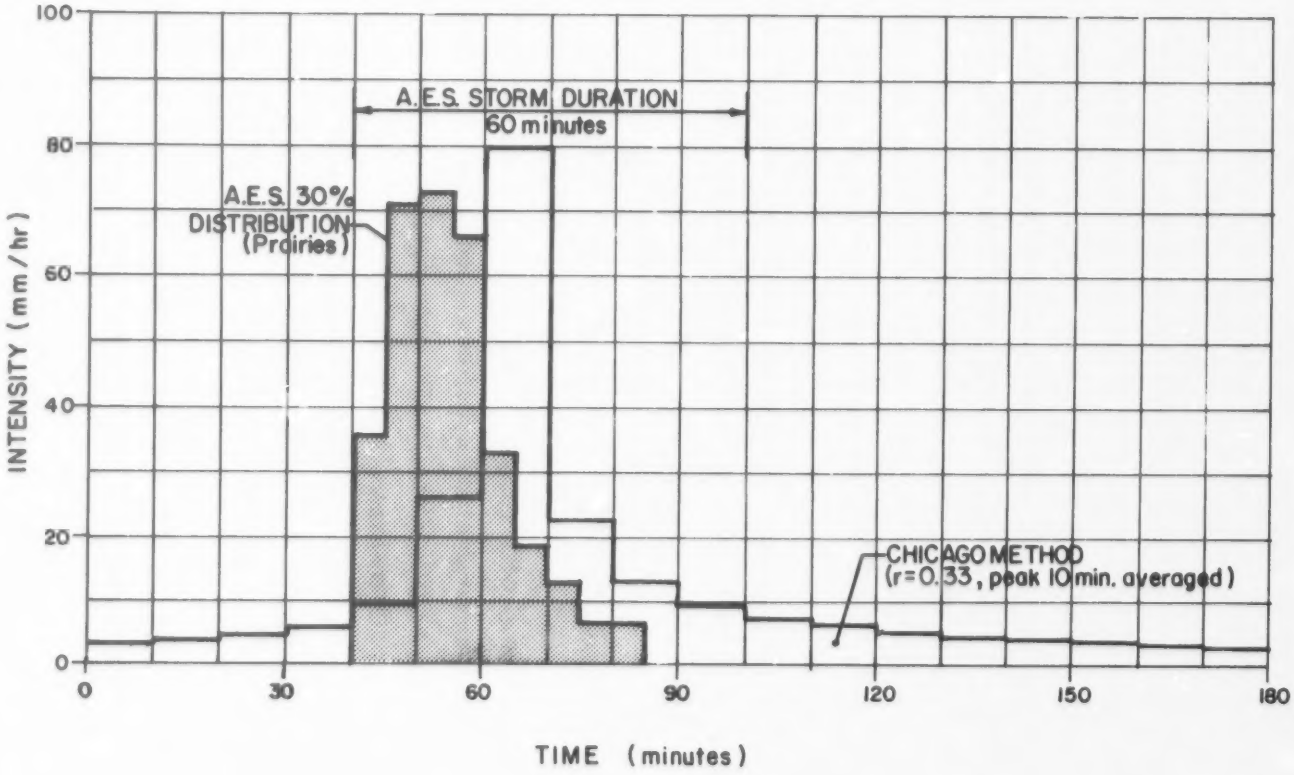
a) the procedure is relatively easy to apply,
b) it contains the critical aspects of storm intensity for all sub-basins within a catchment, and
c) it is conservative.

As indicated earlier, the latter aspect of this procedure has led to considerable criticism being levelled at it. The problem can be related to the PF value of the synthetic design storm (e.g. in Edmonton the 5 year, 5 minute discretization storm has a PF of about 4.1; historic data for the Prairies indicate a value of about 2.7). Adjusting the rainfall intensity during the most intense part of the hyetograph will compensate for this. A uniform rainfall intensity during the peak rainfall period equal to the time of concentration of the smallest sub-basin (usually greater than or equal to 10 minutes) provides a means for reasonable simulation results. The Chicago hyetograph for the Edmonton area, adjusted in this manner, has a PF of 3.0 (Figure 6.2). Hyetographs constructed in this fashion are believed to be reasonable design storm configurations.

Historically Based Design Storms

Use of historical design storms has been common practice for some time. Several such storms have been proposed for use in the City of Edmonton by J.F. McLaren (1979). McPherson (1974) argued strongly in favour of the use of historical storms indicating that a design storm is "a device for facilitating analysis at the expense of credibility". The use of historical storms has public relations value as well. The public can relate better to a system designed to "handle the 1978 rainfall event" better than one that can "handle the 10 year event". As a result, the use of actual historical storm events will continue to find application, particularly in the instance of major system design events.

Recent investigations have attempted to develop design storm distributions that are more realistic than the Chicago hyetograph based on historic storm event data. Research on the time distribution of rainfall has focussed on the cumulative distribution of rainfall within the storm event. From a review of many storms, plots of cumulative rainfall (as a %) versus cumulative time (also, as a %) are made. From these, design storm distributions can be selected. This was done by Huff (1967) for 261 storms in Illinois. From Huff's work, Terstriep and Stall (1974) selected the first quartile (storms with the most rain occurring



DESIGN STORM DISTRIBUTIONS

5 YEAR RETURN PERIOD EVENT

EDMONTON

FIGURE 6.2

in the first 25% of total storm duration) as the most representative of small urban areas. The average first quartile storm distribution is a built-in feature of the ILLUDAS computer model (Section 6.6.3).

There have been other such investigations conducted on design storm distributions. The most relevant for Alberta are the studies conducted by Atmospheric Environment Services (AES) of Environment Canada (Hogg, 1980). Temporal distributions for 35 stations across Canada were developed for storms of 1 hour and 12 hours in duration. In subsequent work (Hogg, 1982) the distributions were given as percentile distributions. Hogg indicated that the 30 percentile distribution is most representative for use in simulation modelling. This conclusion was based on runoff simulations of each historic event and simulations of various design storm distributions and rainfall amounts obtained from IDF curves. Hogg attempted to minimize the discrepancy between the frequency distributions of the peak runoff rates from the historic and synthetic design storms.

In a review of the AES work, Marsalek and Watt (1984) indicated:

"The main advantages of the AES design storms, which are used in urban drainage design, follow from the fact that they are based on actual Canadian data and are available on a nationwide basis. Shortcomings include the lack of guidance for the selection of other storm characteristics (t_d, D), the restriction to two durations, and a possible neglect of the variability of the temporal distributions with return period. Such shortcomings, however, could be overcome with a relatively small additional development effort."

In the above comment, td refers to the timestep for storm discretization and D refers to the duration of the storm event.

6.3.5 Design Storm Duration

In small urban areas, it is important to identify the critical storm duration. For any given system element being designed, some experimentation with various

storm durations should be conducted to determine the critical storm event. Marsalek and Watt (1984) indicated that the storm duration be greater than twice the basin's time of concentration. For most small (up to about 50 ha) urban areas, a storm duration of 1 hour is suitable.

For urban areas where storage is important, longer duration rainfall events are necessary. The duration should be at least long enough that the peak storage volume is reached and the volume of stored water is in recession. The designer must be able to determine that the detention facility will drain within a few days of the storm event. For this purpose, storms of 12 hours to 48 hours duration are required. As these simulations are usually for major system events (e.g. the 100 year event), the temporal distribution of the storm event is not as important.

Experiences with rural runoff models (e.g. HYMO) indicate that with the Chicago hyetograph, the peak flow rate increases as the storm duration increases. This effect is diminished by the time the storm duration reaches 24 hours. As a result, in rural areas, the storm simulation period should be 24 hours (or possibly more where storage is considered). The considerations for time step discretization are similar to those for urban areas (i.e. $\Delta t \approx t_c/2$).

For design storm durations between 1 hour and 12 hours, the temporal distribution can be interpolated from the two AES distributions. The need for more analysis to identify the storm distributions for other durations was identified by Marsalek and Watt (1984). Until the results of such work are available, the use of the 30% AES Prairie distribution in this manner is acceptable.

6.4 RUNOFF ESTIMATION METHODS

6.4.1 Rational Method

The Rational Method is based on an empirical formula relating the peak flow rate to the drainage area, the rainfall intensity and a runoff coefficient. Undoubtedly the Rational Method is the most widely used method of predicting peak runoff rates for the design of urban drainage systems. Its popularity is a direct result of its apparent simplicity and ease of use. However, its simplicity is achieved by lumping of all the complex variables involved in the runoff process into one coefficient. The Rational formula for metric units is:

$$Q = 0.0028\ C\ i\ A\ (m^3/s) \qquad (4)$$

where: C is the runoff coefficient,
i is the rainfall intensity (mm/hr) for a storm of duration equal to t_c,
A is the effective (connected) area of the drainage basin (hectares), and
t_c is the time of concentration for the basin for the particular event (minutes).

The effect of this simplification has been widely discussed in the literature. For example, runoff for a 324 ha watershed computed by 23 designers, varied by 700 percent (RTAC, 1982). This indicates that the underlying assumptions of the method and its limitations are still not widely understood after almost a hundred years of application.

The fundamental assumptions underlying the Rational Method are:

a) The frequency of the runoff is equal to the frequency of the rainfall. This is not necessarily the case for any individual event, an important point when comparing computed values with measured values.

b) The peak discharge at a point is a function of the average rainfall intensity over a duration equal to the time of concentration to the point in question. This assumes that the peak rate of runoff occurs at the point in time when the entire upstream basin is contributing, and the duration of the rainfall equals or exceeds the time of concentration.

c) The rainfall is uniform over the basin and steady with time. Thus, a real distribution of rainfall or the tracking of storms across the basin cannot be directly accounted for.

Concerns regarding the proper application of the Rational Method include:

a) Selection of runoff coefficients is highly subjective, and accounts for much of the variation in the results obtained by this method. Earlier discussion of the rainfall/runoff process shows that the coefficient C is not a constant, but varies with ground cover, soil characteristics, ground slope, depression storage, antecedent rainfall, rainfall intensity and rainfall duration. Many publications list typical Rational C values but few qualify them as to storm type and frequency or soil conditions. Table 6.3, abstracted from RTAC (1982), gives ranges of typical values for urban and rural conditions based on soil types and unfrozen ground conditions. Table 6.4, abstracted from Wright McLaughlin (1967), presents approximate values for specific return period rainfall events.

The potential error in estimating the runoff coefficient increases with the amount of pervious surface in the basin. This is particularly true for higher return period events as indicated in note 2) in Table 6.3 which indicates the values should be increased for storms with return periods greater than 1 in 10 years.

TABLE 6.3
TYPICAL URBAN RUNOFF COEFFICIENTS FOR 5 TO 10 YEAR STORMS

Description		Runoff Coefficient Min.	Mean	Max.
Pavement	asphalt or concrete	0.70	0.83	0.95
Roofs		0.70	0.83	0.95
Business	downtown	0.70	0.83	0.95
	neighbourhood	0.50	0.60	0.70
Industrial	light	0.50	0.65	0.80
	heavy	0.60	0.75	0.90
Residential	single family urban	0.30	0.40	0.50
	multiple, detached	0.40	0.50	0.60
	multiple, attached	0.60	0.68	0.75
	suburban	0.25	0.33	0.40
Apartments		0.50	0.60	0.70
Parks, Cemeteries		0.10	0.18	0.25
Playgrounds		0.20	0.28	0.35
Railroad yards		0.20	0.28	0.35
Unimproved		0.10	0.20	0.30

Notes:
1. Values within the range given depend on the soil type if the watershed is significantly unpaved (sand is minimum, clay is maximum), and on the nature of the development.
2. For storms having return period of more than 10 years, increase the listed values as follows, up to a maximum coefficient of 0.95.
   25 yr. - add 10%
   50 yr. - add 20%
   100 yr. - add 25%
3. The coefficients listed are for unfrozen ground.

Taken from RTAC (1982)

TABLE 6.4

SELECTED RUNOFF COEFFICIENTS AND PERCENT IMPERVIOUS (1)

Land Use or Surface Characteristics	Percent Impervious	Frequency 2	5	10	100
Business:					
Commercial Areas	95	.87	.87	.88	.89
Neighbourhood Areas	70	.60	.65	.70	.80
Residential:					
Single-Family	*	.40	.45	.50	.60
Multi-Unit (detached)	50	.45	.50	.60	.70
Multi-Unit (attached)	70	.60	.65	.70	.80
1/2 Acre Lot or Larger	*	.30	.35	.40	.60
Apartments	70	.65	.70	.70	.80
Industrial:					
Light Areas	80	.71	.72	.76	.82
Heavy Areas	90	.80	.80	.85	.90
Parks, Cemeteries:	7	.10	.10	.35	.60
Playgrounds:	13	.15	.25	.35	.65
Schools:	50	.45	.50	.60	.70
Railroad Yard Areas	40	.40	.45	.50	.60
Undeveloped Areas:		See "Lawns"			
Streets:					
Paved	100	.87	.88	.90	.93
Gravel	13	.15	.25	.35	.65
Drive and Walks:		96	.87	.87	.88
Roofs:	90	.80	.85	.90	.90
Lawns, Sandy Soil:	0	.00	.01	.05	.20
Lawns, Clayey Soil:	0	.05	.10	.20	.40

Notes: 1. From Urban Storm Drainage Criteria Manual (Wright McLaughlin, 1969).

2. These Rational Formula coefficients may not be valid for large basins.

b) It is difficult to determine a realistic value for the time of concentration. This problem is more acute in rural or semi rural basins where there is extensive overland flow across pervious areas. Several methods for estimating the time of concentration are given by RTAC (1982). In urban areas it is common practice to assume an initial inlet time (t_i) in the range of 10 to 20 minutes. However, this is a significantly wide range. For example, using the 1 in 5 year return intensity - frequency - duration curve for the City of Edmonton (i_{10} = 83 mm/hr and i_{20} = 56 mm/h), t_i= 10 minutes yields a peak flow rate 50 percent higher than t_i= 20 minutes.

Downstream, the time of concentration (t_c) is equal to the inital inlet time plus the time of travel (t_t) through the conveyance system. As t_c increases the effect of an error in t_i diminshes, (e.g. at t_c = 20 or 30 minutes the range of peak flow is reduced to 30 percent).

The overall effect of using the incorrect inlet time would be that the system, in particular the upper reaches, could be designed for a storm with a lower or higher return period than was intended. It is imperative that the times of concentration used with the Rational Method are selected with considerable care for each branch of the system.

c) Each reach of the system is designed for the peak flow from a unique theoretical rainfall event over the basin. Thus, the method gives no indication of how the system actually performs during a real event, especially an event that exceeds the design criteria. This is significant for the design of storage facilities, large trunk sewers and in the design of relief works for inadequate drainage systems.

Although the Rational Method has major sources of errors, the potential impact of these errors in the design of small urban drainage systems can be reduced by careful selection of the appropriate parameters for the design event. Opinions as to the size of drainage system which may be designed by the Rational Method

vary widely. The City of Edmonton allows the method to be applied for areas of up to 65 hectares, while other authorities suggest areas between 200 hectares (RTAC, 1982) down to a maximum pipe size of 450 mm (J.F. MacLaren, 1974). It is recommended that use of the Rational Method be limited to systems serving less than 50 ha.

The Rational Method has also been used to estimate storage requirements for stormwater impoundments. This is not a recommended practice as the potential for error is considerable. This is demonstrated by Figure 6.3 abstracted from Winnipeg's Drainage Criteria Manual (J.F. MacLaren, 1974).

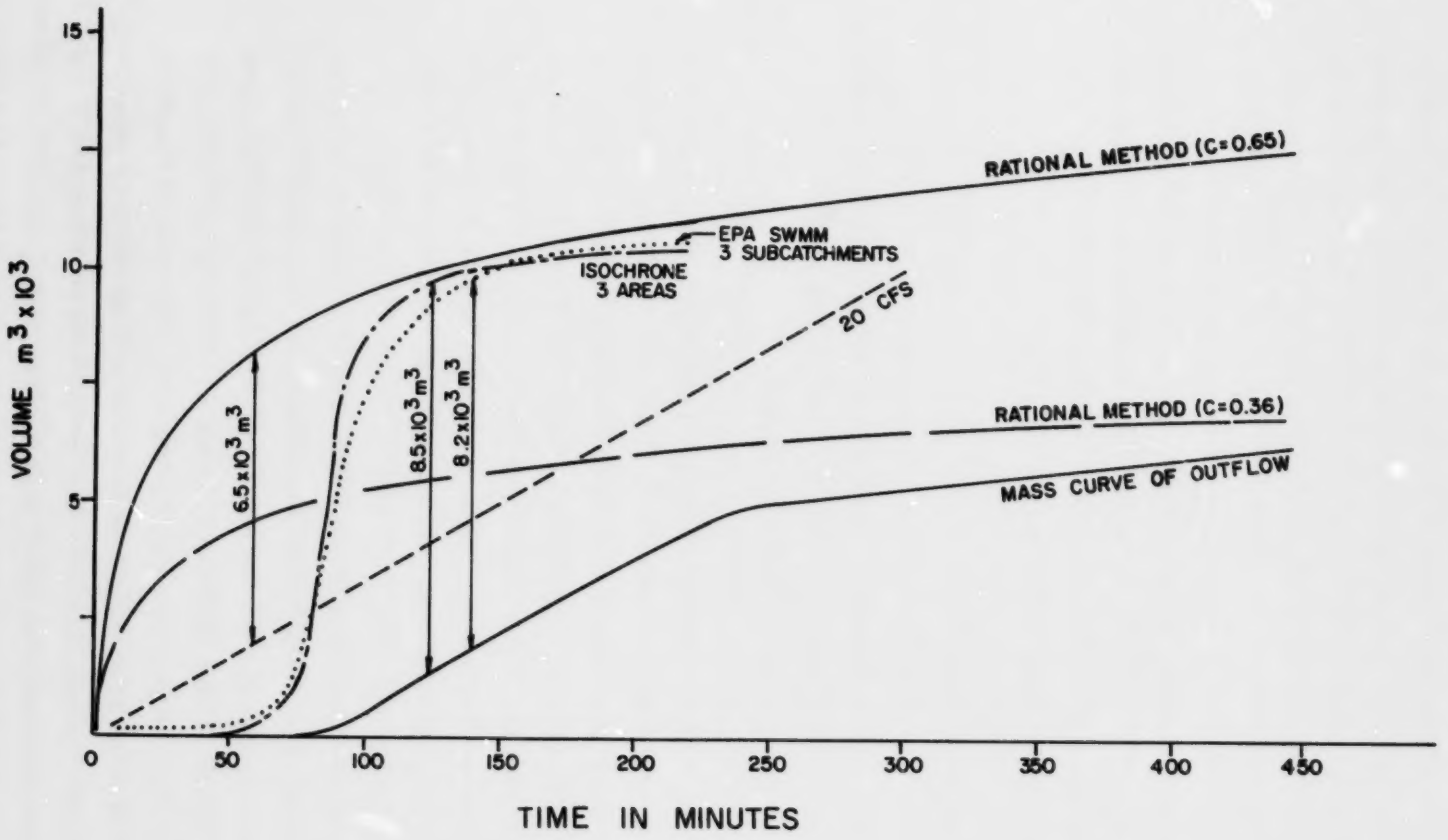
6.4.2 Isochrone Method

The Isochrone Method is a relatively simple way of estimating a runoff hydrograph for an urban catchment. The basis for constructing the hydrograph is a diagram of runoff-time-area and a rainfall hyetograph.

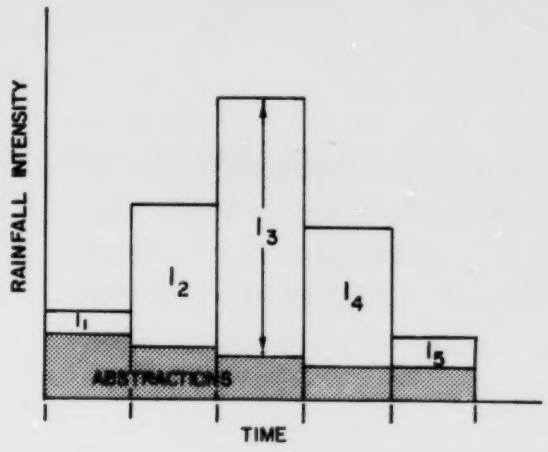
The time-area diagram is constructed by dividing the drainage basin into areas of equal time of travel to the point of reference, Figure 6.4. The time increment used should be the same as that of the design hyetograph. The hydrograph is computed in the manner shown on Figure 6.4, where i is the excess rainfall (after abstractions) at each time step. The method provides a hydrograph that reflects the effects of the rainfall distribution; this is more realistic than an assumption of a triangular hydrograph (a method sometimes used with the Rational Method).

The effects of the varying responses from pervious and impervious areas can be included in the method by developing time-area diagrams and excess rainfall hyetographs for each separately. The excess rainfall hyetographs for impervious areas are obtained by subtracting depression storage and allowing for the effects of surface routing. For the pervious area hyetographs, additional abstractions must be deducted for infiltration using a relationship such as Horton's equation or published or measured values. The individual hydrographs are computed in the manner described above and are then added to give a total hydrograph.

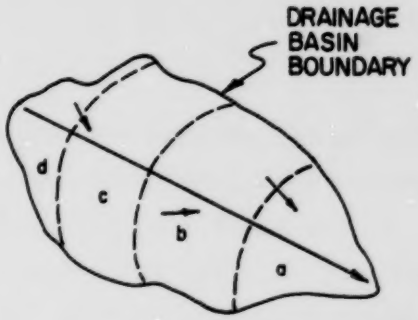
FIGURE 6.3



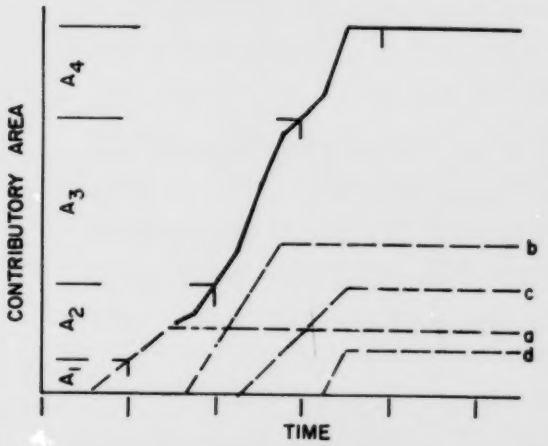
STORAGE BY RATIONAL METHOD



RAIN HYETOGRAPH



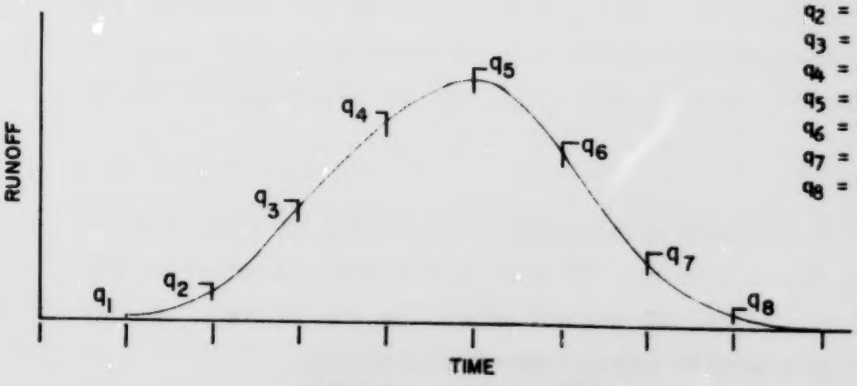
DRAINAGE BASIN



TIME AREA DIAGRAM

Time Area Diagram

- Plot area contributing to outlet versus time for eachsub-basin.
- Add sub-basin curves to get time-area-diag.
- Interpolate incremental areas (A_1 etc) for each time increment.



RUNOFF HYDROGRAPH

Hydrograph

$q_1 = k(A_1 i_1)$
$q_2 = k(A_1 i_2 + A_2 i_1)$
$q_3 = k(A_1 i_3 + A_2 i_2 + A_3 i_1)$
$q_4 = k(A_1 i_4 + A_2 i_3 + A_3 i_2 + A_4 i_1)$
$q_5 = k(A_1 i_5 + A_2 i_4 + A_3 i_3 + A_4 i_2)$
$q_6 = k(A_2 i_5 + A_3 i_4 + A_4 i_3)$
$q_7 = k(A_3 i_5 + A_4 i_4)$
$q_8 = k(A_4 i_5)$

ISOCHRONE METHOD

FIGURE 6.4

This method produces a hydrograph based on a realistic storm pattern which reflects the effects of variations of rainfall abstraction during the storm. The time area diagrams are more easily and reliably computed where a conveyance system exists or has been designed by other methods. It is particulary useful for making preliminary estimates of stormwater storage requirements for urban drainage systems.

The isochrone method does not involve complex calculations and can be carried out by hand for small areas where hydrographs are required at one or two locations. The calculations become time consuming for large areas and/or multiple hydrographs.

6.4.3 SCS Method

This method was originally developed by the Soil Conservation Service, U.S. Department of Agriculture for estimating the runoff from ungauged agricultural drainage basins. The method has been subsequently widely applied to all types of hydrology problems including urban drainage. The validity of such diverse applications has been questioned by numerous sources (Wisner and Cheung, 1979) in particular with respect to urban hydrology.

In this method, the rainfall runoff relationship is expressed in the form of runoff curve numbers (CN). CN has little intrinsic meaning; it is a nonlinear transformation of a watershed storage parameter. In effect, CN relates total runoff to total rainfall for a wide variety of land uses for four hydrologic soil groups (A, B, C & D) and three antecedent moisture conditions (AMC I, II and III). CN values for mixed land uses or soils types can be determined by simple weighting procedures.

The depth of runoff is computed by a simple equation using the total depth of rainfall and CN. CN ranges from zero (0) which will produce no runoff for any rainfall to 100 which produce 100 percent runoff for any rainfall. Runoff hydrographs are then generated by unit hydrograph methodology.

The SCS method can be used in hand calculations for small drainage basins. This is facilitated by charts or nomographs for urban applications published in National Engineering Handbook (USDA, 1972), Urban Hydrology for Small Watersheds (TR55) by the USDA (1975).

6.4.4 eterministic Methods

Deterministic methods quantify runoff from rainfall and/or snowmelt by simulating the effects of the various components of the process. This involves computing the runoff for a number of discrete time steps for the duration of the runoff event. Typically, surface detention storage is first abstracted from the rainfall followed by the abstraction of infiltration on pervious catchments. Infiltration is usually based on a relationship such as Horton's Equation which relates the soil absorption and infiltration capacities with the current rainfall intensity. Next, the excess runoff is routed overland where additional infiltration may be deducted if the water flows over the pervious surfaces. At an appropriate point the pervious and impervious hydrographs are combined and may be further routed through channels to an inlet point.

Deterministic methods involve large amounts of calculation and are much more suited to computer modelling than to hand calculation. None of the existing computer models (discussed in Section 6.6) are purely deterministic although a number of programs are classified as deterministic models. For practical reasons some processes and/or physical characteristics are lumped together and their impact is quantified empirically.

6.4.5 Snowmelt

There are two approaches to estimating snowmelt that are widely used (Gray, 1970), being the 'degree-day' method and the energy budget method.

The 'degree-day' method simply relates snowmelt to mean daily temperature by a coefficient.

$$SM = C\,(T_a - T_b) \qquad (5)$$

where: Sm is the snowmelt in mm/hr,
C is a coefficient,
T_a is the mean daily air temperature, and
T_b is the base temperature above which snow melts.

The energy budget method relates melt rate to a number of atmospheric parameters.

$$M = M_{rs} + M_{rl} + M_{ce} + M_p + M_q \qquad (6)$$

where: M is the snowmelt in mm/hr,
M_{rs} is snowmelt due to shortwave radiation,
M_{rl} is snowmelt due to long wave radiation,
M_{ce} is snowmelt due to condensation and convection,
M_p is snowmelt due to heat content of rain, and
M_q is snowmelt due to heat conduction at ground.

In practice, these equations are difficult to apply for two reasons. Most of the parameters involved have to be estimated, and the equations apply to a uniform snowpack (which is rarely the case). Also, unless recorded snow depth and snow density measurements are available (which is also rarely the case), these also have to be estimated.

For rural catchments the effect of snowmelt can generally be estimated from streamflow records using statistical methods of analysis.

6.5 HYDROGRAPH ROUTING

6.5.1 Hydrologic Routing

The hydrologic approach to flood routing is based on the storage - depth and depth - discharge relationships of natural stream channels. Simply stated, the difference between the inflow and outflow rate at any time is equal to the rate of change of storage in the reach (Chow, 1959). This method assumes that the routed flow is changing slowly with time and that the dynamic effects of flow are negligible. These conditions apply to lakes and to some natural streams.

Numerous hydrologic flood routing methods have been developed based on the hydrologic routing concept. The Puls method for reservoir routing and the Muskingum Method for routing hydrographs on rivers are well known and widely used. However, where streams have steep slopes or where flow rates vary rapidly as in urban drainage basins dynamic effects of flow may be pronounced. Other methods which account for the dynamic effects are noted in reference to various computer programs (Section 6.6).

6.5.2 Unit Hydrograph Methods

The unit hydrograph concept is widely used in hydrology. A unit hydrograph is defined as the hydrograph that would result from one inch of excess rainfall falling over the basin at a uniform rate during a specified period of time or duration (Chow, 1959). A unit hydrograph is derived by analysing recorded hydrographs from a gauged drainage basin and the associated rainfall. Various procedures are available for deriving a unit hydrograph (Chow, 1959). A further development of the method is the concept of instantaneous unit hydrographs which provides a unit hydrograph that is applicable to all storm durations.

By definition, a unit hydrograph represents the routing effects of the physical characteristics of the drainage basin. Thus, a correlation can be made between unit hydrograph parameters (such as a peak discharge, lag time and total base time) and basin characteristics including (area, basin slope, stream density within

the basin, etc.). Using these derived relationships, a unit hydrograph can be transferred from a gauged basin to ungauged basin and be used to predict runoff.

There are numerous unit hydrograph methods, all using varying methods of derivation and differing relationships between hydrographs and basin characteristics, which reflect the hydrological data from which they were derived. These methods can be useful, but particularly in urban areas their validity for application should be verified.

6.5.3 Hydraulic Routing

Flows in storm sewers are generally unsteady and nonuniform when a pipe is not flowing full, and are subject to backwater effects from the downstream end of the pipe. Unsteady free-surface flow in sewers can be represented mathematically by the St. Venant or shallow water wave equations. Expressed in terms of velocity, the St. Venant equations are given by equations 7 and 8; expressed in terms of flow, they are given by equations 9 and 10.

$$\frac{\partial h}{\partial t} + D\frac{\partial V}{\partial x} + V\frac{\partial h}{\partial x} = 0 \qquad (7)$$

$$\frac{1}{g}\frac{\partial V}{\partial t} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{\partial}{\partial x}(h\cos\theta) - S_o + S_f = 0 \qquad (8)$$

dynamic wave

quasi-steady dynamic wave

diffusion wave

kinematic wave

$$\frac{1}{gA}\frac{\partial Q}{\partial t} + \frac{1}{gA}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + \frac{\partial}{\partial x}(h\cos\theta) - S_o + S_f = 0 \qquad (9)$$

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (10)$$

where: V is the flow velocity
Q is the discharge rate,
t is time,
x is the distance along the sewer,
g is the constant for acceleration due to gravity,
h is the flow depth above the invert (measured normal to x),
A is the flow cross sectional area normal to x,
D is the hydraulic depth (which is equal to the ratio of A to water surface width B),
θ is the angle between the sewer axis and horizontal plane,
S_o is sin θ, the sewer slope, and
S_f is the friction slope.

These equations provide a good approximate representation of unsteady sewer flows (Ben Chie Yen, 1977). Solution of these equations for a sewer system are rather complicated, requiring the solution of numerous simultaneous equations or iterative numerical solutions. Some computer simulation models solve the complete equations but these models tend to be expensive to operate. This degree of sophistication is not always warranted with respect to cost and accuracy of simulation. Approximate solutions to the equations are obtained by eliminating various terms in the momentum equation (8 or 9) to produce simpler models as shown.

The quasi-steady dynamic wave approximation neglects only the local acceleration term, however it is less accurate than the diffusion wave approximation. The diffusion wave approximation neglects both of the (first two) inertial terms. However, the retention of the pressure term still permits attenuation of the hydrograph.

The Kinematic wave approximation neglects all but the slope terms in equation (8 or 9). With S_f estimated by Manning's or other formulas the flow is considered as instantaneously steady and uniform. This approximation is widely used in

simulation models as it allows sewer routing to proceed pipe by pipe from the upstream end to the downstream end of a system. However, downstream backwater effects cannot be accounted for using Kinematic wave routing.

The Manning formula equation (11) is widely used to compute the steady state flow capacity for pipes and channels.

$$Q = A * R^{0.667} S^{0.5}/n \qquad (11)$$

where: A is the cross sectional area,
R is the hydraulic radius,
S is the slope, and
n is the friction coefficient.

6.6 COMPUTER MODELS

In recent years, the use of computer models for carrying out hydrologic and hydraulic analysis has increased rapidly. This trend is likely to continue in the future, particularly as the powerful personal computers (micro-computers) now available are capable of using large and complex computer programs.

Numerous computer programs have been developed to model hydrologic and hydraulic systems. Most of these programs were developed for specific systems to provide specific information. Relatively few programs are designed as general application models which can be applied to a wide variety of problems in different locations. Such general application programs tend to require a large computer capacity and extensive input data. These programs also require a considerable level of effort to develop the expertise necessary for their proper use.

A number of programs which are useful in the planning and development of stormwater management systems are discussed below. These programs have been extensively used in Canada. Access to them is usually available at a

nominal cost or no cost at all. However, it should be noted that the costs for the computer resources to make runs varies widely and can be significant.

6.6.1 Stormwater Management Model (SWMM)

This program was developed for the U.S. Environmental Protection Agency and has been publicly available since 1973. SWMM is one of the most comprehensive modelling programs available for urban drainage system analysis. SWMM was developed to provide a common basis for evaluation of pollution abatement options for combined sewer overflows throughout the United States. In order to ensure that the model is applicable to a wide range of climatological and physiological conditions, the hydrologic and hydraulic routines are far more detailed than is normally required for water quality studies.

Because of the sophistication of its hydrologic and hydraulic routines, SWMM has been used extensively to analyse the operation of complex urban drainage systems. The program has received considerable support in Canada through the Ontario Ministry of Environment, the IMPSWM Group at the University of Ottawa, McMaster University, and a number of municipalities and consulting engineering organizations.

The SWMM package consists of six subprograms, called Blocks, which are controlled by a main program call the Executive Block. Although it was intended that the system should operate as one program through the use of overlays, many users find it convenient to operate individual blocks as separate programs. The function of each block is as follows:

Runoff Block

RUNOFF generates runoff hydrographs from a rainfall hyetograph and a physical description of the catchment areas. Simulated runoff is routed overland using the kinematic wave method. RUNOFF also generates pollutographs for the simulated rainfall event and user input pollutant loadings.

The program generates runoff independently for three hypothetical sheets representing the catchment surface.

Sheet 1 is an impervious surface with no depression storage. It therefore produces immediate runoff. Without this component no runoff would be simulated until the accumulated rainfall exceeds the total depression storage for the pervious area, an unrealistic condition.

Sheet 2 is an impervious surface from which all the impervious area depression storage specified by the user is abstracted before runoff occurs. The relative sizes of sheet 1 and sheet 2 are user specified.

Sheet 3 is the pervious surface from which depression storage and infiltration are abstracted. Infiltration is computed by either the Horton equation or the Green-Ampt equation as selected by the user. The program keeps track of both the accumulated rainfall and accumulated infiltration as well as the instantaneous rainfall and infiltration. It routes the excess rainfall across the pervious sheet and continues to infiltrate surface water in transit if there is excess infiltration capacity. Thus, if the rainfall hyetograph contains a very intense rainfall time step followed by a low intensity time step, additional water will be abstracted during the second time step. It would be abstracted from that portion of the excess rainfall which did not flow off the pervious sheet during the preceding time step. While this represents a realistic simulation, it is a facility which can significantly effect results as discussed below.

The program adds the discharge from the three sheets together at the discharge node. This node can represent the inlet point to a sewer system, for subsequent pipe routing by the TRANSPORT or EXTRAN BLOCKS, or the inlet point to a gutter or open channel through which RUNOFF can simulate additional routing.

All of the input variables for the RUNOFF BLOCK are deterministic, except for W, the width parameter. This parameter determines the shape of the catchment area. If W is relatively large with respect to the catchment area, a short overland flow route to the inlet will be simulated. Conversely, if a relatively

small value of W is used, a long overland flow route to the inlet will be simulated. The effect of W is the same for all three flow sheets. The value of W can affect not only the shape of the simulated runoff hydrograph but can also affect the volume as runoff due to the infiltration simulation on sheet 3.

The SWMM manual suggests setting W equal to twice the length of gutter in the catchment as an initial estimate (which should be refined by calibration). The validity of this assumption will vary with the degree to which the drainage system is discretized and the physical layout of the catchment area. The impact of this parameter is often overlooked when calibrating a RUNOFF model.

Transport Block

TRANSPORT simulates free surface (open channel) flow of runoff through a drainage conveyance system of pipes or channels. Input data to TRANSPORT are hydrographs and pollutographs generated by the RUNOFF BLOCK (which are stored on a transfer file) and data describing the conveyance system.

The conveyance system is represented as a series of links and nodes typically representing the pipes and manholes of a sewer system. A number of other sewer elements including weirs, diversions (branched sewers), pump stations and storage facilities can be simulated as special types of nodes. A number of common pipe and channel cross sections are available within the program, and unusual cross sections can be user defined.

In practical applications, it is not feasible to simulate individual pipes and manholes due to the large computer memory requirements this would entail. Typically, the nodes are used to represent junctions where significant changes in flow occur, conduit sizes change, or where the conduit slopes change. The Users Manual warns that there is an upper limit to the length of a link of about 1000 m required to maintain computational accuracy. There is also a lower limit to the length of a link. The link must be long enough so that water cannot flow from

the upstream to the downstream end of the link (simulated conduit) in a shorter time than the computational time step specified for the simulation. If this situation occurs, water will be lost from the system.

TRANSPORT cannot simulate surcharged or pressurized flow in sewer systems. If the inflow at a point exceeds the 'full capacity' of the downstream conduit, the excess water is stored at the upstream node until the rate of flow recedes to the point where there is excess downstream capacity. The stored water is then returned to the system.

The flow routing routine used in TRANSPORT is quite sophisticated. The program approximates the solution of the St. Venant equations representing gradually varied, unsteady flow conditions in a conduit, using an explicit solution technique. Computations of flow conditions are made for all elements in the model at each time step, starting at the upstream ends and working progressively downstream.

Downstream effects are only approximated in TRANSPORT. This is not normally a problem except where pipes are steep enough to cause supercritical flows to occur. In such cases flows may be translated through the conduit without any routing. Also, where storage elements are used in the model, including "super-pipes", the simulated output may imply backwater conditions upstream of the storage element which are false.

TRANSPORT has the facility to increase conduit sizes where simulated flows exceed the free flow capacity of the system. This facility can be used as a design aid for developing systems, particularly at the planning stage.

Extran Block

EXTRAN is a more sophisticated pipe routing program than the TRANSPORT BLOCK. It was specifically designed to model complex sewer systems. EXTRAN can simulate backwater conditions, looped pipes, flow reversals and surcharged or pressure flow conditions. This enchanced ability to simulate

complex systems is achieved by a methodology that solves the complete St. Venant equations.

Originally, EXTRAN contained water quality routing routines which have subsequently been removed as they were rarely used. The program has been used extensively for hydraulic analysis of complex sewer systems, particularly for projects involving flood relief of combined sewer systems. Poor documentation compared to other SWMM BLOCKS, combined with the level of effort required to develop the expertise to use the program, however, appears to have limited its use.

EXTRAN is far less 'user tolerant' than other SWMM BLOCKS due to the potential for mathematical instabilities to occur. Constraints on pipe lengths, slopes and specifications of special elements (weirs, flap gates, etc.) are more demanding. EXTRAN requires a very small computational time step, generally 5 to 20 seconds, resulting in a large number of calculation steps. Since it is impractical to output the results of each time step, numerical instabilities can propagate through the system between output cycles (which are generally at 5 minute intervals or longer). This makes it difficult to determine the source of such problems as the effects can appear both upstream and downstream of the source. One approach to minimize such problems is to develop and test large models in small segments.

The ability of EXTRAN to simulate surcharged flow was the principal feature which attracted initial users of the program. Unfortunately, early versions of the program were able to produce significant errors under heavily surcharged flow conditions. The solution technique uses the change in the in-system storage (physical pipe volume) over a time step to maintain continuity in the simulation process. In order to maintain numerical stability in the transition from in-pipe flow (relatively large storage) to surcharged flow (minimal storage at manholes), "artifical storage" is introduced to produce a smooth transition. In the early versions of the program, "artificial storage" was related to the system geometrics and could become excessively large. This would result in excessive attenuation of the discharge hydrograph, and consistent underestimation of

surcharge levels. This problem has apparently been controlled in the most recent version, SWMM3 (1983). However, users are cautioned to critically examine the results for heavily surcharged systems.

Storage/Treatment Block

The STORAGE/TREATMENT BLOCK simulates the impact of storage on the quality of stormwater effluent.

Receive Block

The RECEIVE BLOCK is used to simulate the impact of the quality of stormwater effluent on the receiving stream.

Combine Block

The COMBINE BLOCK is a utility routine which facilitates the modelling of systems too large to model as a single system. The COMBINE BLOCK can be used in a number of ways including collating data sets and/or combining data sets from one model run for input into another model.

SWMM - Derivatives

There are a number of modified versions of SWMM developed by various organizations. Of particular interest are the following:

CANSWMM - Canadian SWMM

This was an adaptation of the early version of SWMM for Canadian conditions developed for Ontario Environment. Snowmelt routines were first incorporated in this version. Of particular interest in the development of this version was the application of the model to studies of water quality in receiving streams or water bodies.

OTTSWMM - Developed by the IMPSWM Group of the University of Ottawa

In this version the RUNOFF BLOCK has been modified to allow simulation of a dual drainage system (minor and major). This program is widely used in Ontario in the design of new drainage systems.

PCSWMM - Developed by Computational Hydraulics Incorporated

This package is the latest version released by the EPA, SWMM3, converted to run on IBM-PC and compatible micro-computers. Apparently the conversion has been effected without any significant modifications to the computational code. Input and output have been modified to make it screen-orientated. The package includes an interactive preprocessor to facilitate the assembly of input data, including some error checking capabilities, and a post processor statistical package. The post processor is designed to facilitate interpretation of continuous water quality simulations.

Execution time is considerably longer for a micro-computer compared to a mainframe. However, processing time is usually a relatively small component of the total time involved in modelling studies.

6.6.2 Hydrograph Volume Method (HVM)

In Canada, the HVM was first used in a number of stormwater management studies in Toronto in 1970. The model was subsequently used in Vancouver in 1974 as part of a combined sewer separation program in the City's West End. This model was the first commercially available stormwater model that could deal with conduit surcharging. However, the HVM model was the property of Dorsch Consult Limited of West Germany. In the initial decade of its usage, HVM could only be used by retaining Dorsch as a consultant. This restricted the usage and interest in the model considerably. As a result, SWMM with EXTRAN have enjoyed greater popularity. Recently, the HVM has been made publicly available on a time-share basis.

The HVM model comprises five programs which can be run together or separately. The interfacing of data between programs is accomplished by means of an Additional Data Tape. The Additional Data Tape is essentially a common formatted file for storing the output of the individually run programs (Partial Fill Curves, Model Storm and Surface Runoff) prior to the execution of the Data Editing Program and finally the Sewer Network Flow Model. These programs are described briefly in the following sections:

Partial Fill Curves Program

The Partial Fill Curves Program determines several hydraulic elements for the various types of system conduits (including open channels) as a function of depth. The hydraulic elements are:

a) cross-sectional area,
b) surface width,
c) hydraulic radius,
d) flow rate, and
e) flow velocity.

For flow depth greater than 50%, the program adjusts for the influence of air friction in partially full conduits.

Each typical geometric shape is analysed and the computed attributes are stored on the Additional Data Tape. These hydraulic characteristics are referenced whenever that particular type of conduit is encountered in the Sewer Network Flow Model.

Model Storm Program

The Model Storm Program develops a synthetic design storm based on the Chicago method, originally developed by Keifer and Chu (1957). Input to the model are the Intensity-Duration-Frequency curve parameters (see section 6.3.2), the degree of advancedness of the peak rainfall intensity (which is the

time to peak intensity divided by the total storm duration), the storm duration and the time step for the storm discretization. Output from the program is stored on the Additional Data Tape. In the newer metric version of HVM, this model is a subroutine of the Surface Runoff Model.

Surface Runoff Model

The Surface Runoff Model transforms the rainfall hyetograph (either a real event, a separately derived design storm, or the synthetic design storm from the Model Storm Program) into inflow hydrographs at a catchment's inlet point. For each surface type within the study area, the nature (i.e. length, slope, roughness and detention depth) of the roof, paved and green areas are specified. The split between roof and paved areas and the domestic or dry weather flows are also identified here. The continuity and energy equations are solved to develop the specific runoff hydrographs for each surface type. Infiltration is accounted for using Horton's equation.

These typical surface type hydrographs are stored on the Additional Data Tape for hydrograph construction in the Sewer Network Flow Model. When a sub-basin is identified in the Sewer Network Flow Model, only the surface type code, the area and the percent imperviousness are needed.

Data Edit Program

The majority of the computer resource cost in using the HVM package is incurred by the Sewer Network Flow Model. The Data Edit Program is an error checking facility which examines and flags any errors in the Additional Data Tape and the data file for the Sewer Network Flow Model.

Sewer Network Flow Model

The Sewer Network Flow Model simulates the dynamics of the stormwater management system during the storm event. The model simulates closed

conduits, open channels, overflows, detention facilities and diverging conduits. Up to 1100 conduits and special structures can be simulated in one run.

The basis of the HVM is an iterative solution to the St. Venant energy and continuity equations in finite difference form at each time step. The program has a significant advantage over SWMM's EXTRAN BLOCK in that the Courant condition does not have to be satisfied to achieve computational stability (i.e. a 5 minute time step is acceptable). The program fulfills its mathematical requirements by assuming that all of the upper ends of the system have no inflow and that all outfalls have an HGL which is a function of flow. A user can also specify the inflows and HGLs as functions of time by putting this information on the Additional Data Tape. In this way, hydrograph takeover from upstream systems or backwater conditions at outfalls can be simulated.

In its calculation procedure, HVM takes the surface runoff hydrograph for each conduit and forces the flow to enter the system along the length of the conduit. This is done regardless of the hydraulic gradeline elevations that might occur in the conduit system. This representation is realistic as long as the extent of simulated conduit surcharging does not greatly exceed ground level (in HVM, ground level cannot be specified as an HGL constraint at a manhole). For events where the conduit system capacity is greatly exceeded and substantial surcharging above ground level is simulated, the hydraulics are not realistic. Peak flow rates for the conduits will be greater than those that will actually occur. This shortcoming does not preclude the use of the model for relief sewer planning (where conduit system capacity will be provided to accommodate a design event). However, use of HVM for major drainage system planning requires a considerable amount of judgement.

6.6.3 Illinois Urban Drainage Area Simulator (ILLUDAS)

The ILLUDAS model was developed as a tool to facilitate the design of urban drainage systems. It combines both hydrologic (runoff) computations and hydraulic (pipe routing) analysis in one model.

The hydrologic computations are based on the Isochrone method (reviewed in Section 6.4.2). Runoff hydrographs from impervious, supplementary impervious and pervious areas are computed separately. The supplementary impervious hydrograph is added to the pervious surface hydrograph prior to the computation of infiltration losses. The combined hydrograph is added to the impervious area hydrograph at the inlet point. Although infiltration is computed by the Holton Equation, user options are limited to selecting one of the four Soil Conservation System hydrologic soil groups and one of four antecedent moisture conditions. Depression storage estimates for both pervious and impervious surfaces are user defined.

A simple storage routing technique is used to simulate flow through pipe or channel sections in the drainage network. The technique uses storage discharge relationships computed by the Manning formula and a simple storage routing formula. Complete hydrographs are routed through each reach in succession. Backwater conditions are not simulated in ILLUDAS.

The model temporarily stores runoff in excess of each pipe capacity at the upstream pipe node and can output nodal storage volumes. This feature is useful for determining preliminary storage requirements in the design of stormwater storage facilities.

6.6.4 Hydrologic Model (HYMO)

The HYMO program was developed by the U.S. Department of Agriculture in 1973 and is described as a problem orientated computer language for modelling surface runoff and sediment yield. The program is designed to be highly interactive allowing the user to carry out a step by step analysis using a set of command words. The model can also be operated from an input file of commands and data. These commands allow the user to compute a runoff hydrograph, compute a rating curve for a channel reach, compute a travel time table for a channel reach, and route a hydrograph through a channel reach or a reservoir. Additional commands enable the user to store and retrieve hydrographs and rating tables, add hydrographs together and print or plot output.

Rainfall input to the model is in the form of a mass curve instead of the more usual hyetograph format. The program can only retain six hydrographs in memory at a time, a minor limitation for large systems which can be overcome with careful sequencing of the analysis steps.

Channel routing is carried out using the Variable Storage Co-efficient method with modifications to account for changing water surface slope during a flood. HYMO uses the storage-indication method to route floods through reservoirs. These routing capabilities are superior to those used in ILLUDAS based on studies carried out by the IMPSWM Group.

Hydrographs are generated using the SCS method to determine the rainfall excess and a unit hydrograph developed by the program authors (Williams and Hann, 1973). While this approach provides acceptable results for a wide variety of rural drainage systems, it does not produce accurate hydrographs for urban catchments (see Section 6.6.6).

The simplicity of this model in terms of both user modelling expertise and data requirements make it an attractive model for drainage planning purposes, and led the IMPSWM Group to develop additional procedures to model urban drainage components.

OTTHYMO

The OTTHYMO program contains all of the original HYMO commands plus three additional commands (URBHYD, NASHYD and KINROUTE) developed by the IMPSWM Group.

The URBHYD routine is designed to produce results consistent with those produced by the RUNOFF BLOCK (SWMM) for large lumped catchment areas. URBHYD uses essentially the same hydrologic input data as RUNOFF. Where the URBHYD and RUNOFF routines essentially differ is in their hydrograph routing procedures. URBHYD uses unit hydrographs derived from a synthetic linear reservoir system concept to simulate the lag effect of overland flows.

Runoff hydrographs from pervious and impervious surfaces are computed separately using different reservoir systems.

The NASHYD routine gives the user the option of using either the NASH unit hydrograph or the Williams and Hann unit hydrograph originally used in HYMO. The NASH unit hydrograph produces a shorter recession limb for the output hydrograph which may be more appropriate for small rural watersheds. In addition, the user can specify the initial rainfall abstraction instead of using the fixed relationship in the SCS method in HYMO. The initial rainfall abstraction can be based on the Antecedent Precipitation Index (see section 6.3.2).

Rainfall input for OTTHYMO is in the form of a hyetograph, making it compatible with most other urban drainage models.

KINROUTE is a routine for routing flows through pipe systems based on the diffusive kinematic wave model. Validation of the model (Wisner and P'ng, 1982) indicated that results compared favourably with sophisticated dynamic models for free surface flow conditions. The entire hydrograph is routed through a pipe section before proceeding downstream. This is necessary to conform with the basic operation of the HYMO program. Backwater effects are not simulated.

6.6.5 Hydrologic Engineering Centre Programs

The Hydrologic Engineering Centre, a section of the Corps of Engineers, U.S. Army, has developed and continues to maintain a number of comprehensive programs for modelling hydrologic engineering problems. In particular the programs STORM, HEC-1 and HEC-2 may be useful in some stormwater management applications.

STORM

The STORM program is primarily a water quality model capable of continuous simulation of runoff events and pollutant loadings from hourly precipitation records. The hydrologic routines in the program are relatively simple. If the

average daily temperature is below a specified threshold value, precipitation is accumulated in a snowpack. If the average daily temperature is above the threshold value precipitation is treated as rainfall and/or the residual snowpack is melted at a rate computed by the degree day method.

Two methods are available for computing runoff. A coefficient method which assumes a constant ratio of runoff to rainfall minus depression storage. Recovery of depression storage, from a specified maximum value, is computed continuously from an input average evaporation rate. This method is more suitable for highly impervious drainage areas than for pervious areas.

The second method is the SCS Curve Number Technique with provision to recover infiltration and detention storage capacity during dry periods. This method is more suitable for pervious drainage areas. The program will simulate dry weather flow for combined sewer systems. A triangular unit hydrograph concept is used for routing flows through the drainage basin.

The water quality routines allow STORM to simulate the accumulation of pollutants on the drainage basin during dry weather, the pollutant wash-off during runoff events and the impact of overflow diversions, in-system storage and treatment on the discharge of pollutants to a receiving water body.

As a hydrologic model, STORM is not suitable for simulating high intensity short duration rainfall events which typically control the design of many urban drainage systems. However, it has been used to search historic rainfall records to identify historic high runoff events or high runoff periods.

HEC-1

The HEC-1 program is described as a flood hydrograph package which can model runoff from precipitation on a complex, multisubbasin, multichannel river basin. The program is limited to analysing single events as there is no provision for recovery of rainfall abstraction rates during dry periods.

Snowmelt can be computed using either the degree-day method or the energy budget method. Rainfall losses (infiltration and depression storage) are computed by a function relating loss rates to rainfall intensity and accumulated loss. The program will compute a unit hydrograph or will apply a user supplied unit hydrograph. Channel routing can be carried out using one of six optional hydrologic procedures (Modified Puls, Muskingum, Working R and D, Straddle - Stagger, Tatum and Multiple Storage).

An unusual and valuable feature of the program is that it will derive loss rate and unit hydrograph coefficients or routing coefficients from a recorded hydrograph. A procedure to use this facility to calibrate a system model is described in the program users' manual.

The program also has the facility to evaluate alternative scenarios in terms of floodplain damage - flow frequency relationships. Scenarios can include existing conditions, changes in land use, increased in-system storage, or improved channelization. Comparisons are made on the basis of estimated average annual damage costs.

A version of HEC-1 is available which will operate on an IBM PC and compatible micro-computers. However, the economic comparison routines are not included in this version.

HEC-2

The HEC-2 program computes the water surface profile for river channels of any cross section for either subcritical or supercritical flow conditions. The effects of bridges, culverts, weirs and other hydraulic structures can be modelled.

Storage discharge curves for river reaches can be transferred to HEC-1 for subsequent stream routing analyses. This program is widely used to compute backwater profiles in rivers and open channels and for floodplain delineation. A version of HEC-2 is now available that will operate on a micro-computer.

6.6.6 Model Calibration and Verification

Virtually all documentation and references for stormwater modelling programs emphasize the need for calibrating and verifying models developed for each system studied. Unfortunately, this is impractical in most cases due to a lack of measured flow data, especially for urban drainage systems. Data is more likely to be available to calibrate rural drainage systems, as a considerable number of small streams in Alberta are gauged. However, even these data are of limited value for calibration exercises as there is rarely adequate rainfall monitoring within these watersheds.

In the past, short term flow monitoring programs for model calibration were regularly included as part of urban drainage system studies. In retrospect, few of these programs could be considered successful as they produced little useful information in relation to the cost and time expended. There are a number of reasons for this, including:

a) There are relatively few locations within sewer systems where reliable measurements can be obtained.
b) Robust and expensive equipment is required to withstand the harsh sewer environment (constant maintenance is required).
c) The contribution of the pervious areas during the more common events is small; as a result, little knowledge of how these areas behave during extreme events is obtained.
d) Many of the systems monitored in the past were flood prone and were flooded during larger runoff events (thus, measured flows were attenuated to an unknown extent; often the equipment was damaged due to surcharging or other problems).

The overall lack of success of such programs led to the concept of test catchments which would be highly instrumented and monitored on a long term basis; however, few such projects have materialized. One exception is the City of Edmonton which has had an extensive flow and rainfall monitoring program in operation for several years. Other municipalities should endeavor to implement

rainfall/runoff monitoring to facilitate model calibration. In time, such programs will provide a better insight into the hydrologic characteristics of urban drainage systems.

With little or no calibration data available, deterministic models such as SWMM have generally been acknowledged to be the best representation of the rainfall/runoff phenomena in urban systems. The IMPSWM Group, for example, used SWMM as a benchmark for determining the acceptablity or reliability of other models. The user, however, must exercise a considerable degree of judgement in choosing parameter values and should examine the effect of each variable on the simulated results.

Irrespective of the sophistication of a computer model, it is possible to generate significantly different results for the same project with seemingly minor variations in the input data. It is therefore reasonable for local authorities to require those conducting model studies to justify the results obtained by their models. An indication of the sensitivity of the results should be given.

7.0 WATER QUALITY CONSIDERATIONS

7.1 GENERAL

Historically, there has been a tendency to regard stormwater as a relatively minor pollution source - a nuisance rather than a real problem. Numerous studies since about 1970, however, have noted the significance of stormwater pollution. Urban runoff can have characteristics similar to raw sewage. Urban runoff is usually high in suspended solids and biochemical oxygen demand (BOD); it can contribute significant concentrations of toxic metals, salts, nutrients, oils and grease, bacteria and other contaminants. Stormwater discharges to receiving waters can thus have significant impacts on potable water supply, aquatic habitat, recreation, agriculture and aesthetics.

The natural environment has some ability to mitigate the impacts of pollution. Relatively small urban areas draining into large lakes or rivers are unlikely to cause significant effects. If, however, an outfall is located close to a beach, a domestic water supply intake or a biologically sensitive area, it may be a significant pollution source. The use made of the receiving water is the main factor to be considered when evaluating the significance of an urban stormwater discharge.

In evaluating stormwater discharges from an urban environment, the cumulative impacts of future development and the background pollutant loads in the receiving water should be considered. While one outfall may not pose a significant problem, it can set a precedent allowing the future construction of outfalls to the point where an undesirable situation results. It is likely that little can be done to correct the problem when it does become apparent, since retro-fit techniques are often prohibitively expensive. This situation is no different from that associated with increased flows and flooding, and provides further support for the establishment of Watershed Drainage Plans and Master Drainage Plans for developing areas.

The presence of other sources of pollution must also be considered. If a lake or river is exhibiting signs of distress as a result of domestic, agricultural or industrial pollution, it may be undesirable to introduce additional pollutant loads unless they are very small compared to the background concentrations.

Generally, urban stormwater is a controllable source of pollution. As a minimum, treatment in the form of sediment control should be encouraged where feasible. Proponents of stormwater drainage systems in Alberta are advised that treatment in addition to sediment control may be required in cases where water quality impacts on the receiving watercourse are of particular concern. Furthermore, this requirement may also lead to a regulated monitoring program in particularly sensitive watersheds.

7.2 WATER QUALITY CHARACTERISTICS OF STORMWATER

The chemical make-up of stormwater is primarily dependent on the land use within the catchment and the location of the urban area with respect to sources of atmospheric pollution such as major industries or other large developments. Stormwater runoff can contain a wide variety of contaminants, often at concentrations substantially exceeding water quality guidelines and criteria. In addition, changes to the general water chemistry caused or aggravated by stormwater can reduce the efficiency of water treatment plants.

In evaluating the significance of stormwater pollution, both the loadings and the concentrations of contaminants must be considered. Loadings are usually expressed in kilograms per year. Since stormwater pollution events are random in nature (depending on runoff intensity, inter-event times and climatic variables), consideration must be given to the magnitude and distribution of loads over a number of years. The characteristics of a receiving water must be evaluated in terms of its ability to assimilate and dilute the loads imposed upon it. Concentrations are important for this, as the assimilation must be evaluated in terms of receiving stream objectives (expressed as concentrations).

It is difficult to relate concentration and loading data from other studies to another specific study only. As concentrations are the only way water quality can be measured, Table 7.1 is useful as an indication of actual measured stormwater quality. Total annual loading data are affected by annual precipitation and intensity characteristics. Since most references normalize their data by area and land use, but not by precipitation, there is a wide variation of concentrations and loads reported in the literature. Extrapolation of data from one study area to another should only be done by individuals who are familiar with the water quality characteristics of urban runoff.

7.3 THE NEED FOR WATER QUALITY ANALYSIS

Analysis of the quality of urban runoff remains almost as much an art as a science. Rigorous analysis requires the collection and assessment of a great deal of data - an expensive and time consuming undertaking. It is only feasible to conduct such a thorough analysis for the most critical problems. Simple and approximate alternatives must be employed to address most situations, recognizing that the results will be subject to considerable uncertainty.

Hand calculation methods are available for the estimation of single event and annual loading for some parameters such as suspended solids, biochemical oxygen demand, phosphorous and nitrogen. These methods are based on the results of regression analyses conducted in the U.S.

The need for water quality analysis should be considered in two broad categories. First, if a municipal water supply, recreational area or particularly sensitive biological resource is likely to be affected, there is need for a fairly comprehensive analysis. Similarly, if there is potential to significantly aggravate an existing water quality problem, such analysis may be justified. Detailed study will likely include several years of data collection and computer analysis to estimate the magnitude and frequency of loadings and their ultimate fate. This will allow evaluation of the effects of various control or treatment measures, such as sewer diversion or detention ponds.

TABLE 7.1

URBAN RUNOFF CHARACTERISTICS

Parameter	Seattle[1] Washington	Lake Ellyn[2] Michigan	Peak Conc[3] USA	ASWQO[4]
Conductivity, uohm/cm	12.9			
Turbidity, JTU	7.0			25
DO, mg/l	9.0			5.0
BOD, mg/l	30.4	18.0		
COD, mg/l	99.0			
Chloride, mg/l	11.6	34.7		
Sulfate, mg/l	20.0			
Nitrogen, mg/l				1.0
Organic	1.71			
Ammonia	0.35	0.18		
Nitrite	0.13			
Nitrate	0.74			
Phosphorus, mg/l				.15
Hydrolyzable	0.36	0.08		
Ortho	0.11			
Lead, ug/l	360	224	460	50
Iron, mg/l	1.99			0.3
Mercury, ug/l	0.17			0.1
Arsenic ug/l	-		50.5	10
Copper, ug/l	-	41	100	20
Cadmium, ug/l	15.0		14	10
Zinc, ug/l	120	171	2 400	50
Phenols, ug/l			115	5
a-BHC, ug/l			0.1	
Solids, mg/l				
Settleable	121			
Suspended	160	196		Background + 10
TDS, mg/l	144			
Coliforms, org./100ml				
Total	26 000			2400
Fecal	1 200			200

Notes:

1. From Kibler (1982).
2. From Hey and Schaefer (1984).
3. From Cole et al (1984).
4. From Alberta Environment (1977).

The second category comprises projects which would, at first glance, appear innocuous but whose cumulative effects could be of concern. The scale of most land development projects in Alberta is too small to cause substantial water quality impacts by themselves or to justify the cost of extensive water quality studies. The potential for serious problems to arise due to the cumulative impact of multiple developments should be identified at the River Basin Planning or Watershed Drainage Plan levels. Consideration should be given to both the costs of maintaining water quality and the risks associated with making a wrong decision, recognizing the high cost of future remedial measures.

Most potential water quality problems fall into the second category where the uses of the receiving water are not vital and there is no evidence of immediate serious water quality degradation. In such instances, the use of stormwater ponds for water quality control in new developments should still be encouraged, if economically feasible. If in these instances, ponds will be required for flood or downstream erosion control, their benefits in improving water quality will be ancilliary and will involve only a small additional expense. By trapping sediment, such ponds capture in the order of 50 to 90% of most contaminants contained in stormwater. As most contaminants in stormwater runoff are associated with sediment, the design of such facilities must allow for the accumulation and removal of suspended solids to be effective. To also keep the water quality of the pond high it is desirable to capture the sediment before it enters the pond.

7.4 METHODS OF WATER QUALITY ANALYSIS

7.4.1 Sedimentation

Where receiving water uses are not critical, a sedimentation facility will generally provide an adequate measure of water quality protection for the receiving stream. In cases where receiving water considerations are important, it is necessary to consider the particle size distribution for soils in the development area, together with data on the gradation of particles carried by stormwater. The distribution of particle sizes will be influenced by the relative

inputs of local soil erosion, dustfall, pavement particles pulverized by traffic and materials dumped on the roads in winter. Although the relative distribution of these inputs is seasonal and dependent on location, some amount of data should be collected and used as an input to the design of the sedimentation facility.

The design of a sedimentation facility also requires knowledge of the amount of influent sediment. The frequency of maintenance can be roughly determined from the expected rate of sediment accumulation estimated using either computer or more approximate hand calculations. Computer calculations (e.g. STORM, SWMM) are preferable since several years of precipitation records should be analysed. The type and frequency of street maintenance will affect the total sediment load, but has less impact on the transport of other contaminants. Contaminants tend to be associated with the finer size fraction of the sediment (which is not effectively removed by street sweeping).

7.4.2 Water Quality Simulation

Data Collection

In situations where the water quality impacts on the receiving watercourse are of particular concern, a water quality sampling and simulation study should be conducted. Such a study can require several years for the compilation of data and a significant budget for the analysis of the data. Several years of data collection is necessary because of the extreme range of environmental conditions which can be encountered; short periods of data collection do not yield sufficient data for model calibration and verification.

Site-specific tipping-bucket rainfall data, runoff data, and contaminant concentration data must be collected. The range of contaminants to be sampled depends on the end use of the receiving water, but should generally include at least the following parameters:

a) Total Suspended Solids (TSS),
b) Total Dissolved Solids (TDS),
c) Biochemical Oxygen Demand (BOD),
d) Total Phosphorus (TP),
e) Total Nitrogen (TN),
f) Lead (Pb),
g) Zinc (Zn),
h) Total Coliforms, and
i) Fecal Coliforms.

The sampling program should be conducted on catchments having a similar land use adjacent to the area of planned development. The program must capture data for all runoff events (large and small) during the study period. Complete runoff events must be sampled. Missing data for contaminant washoff during the earlier part of the storms will prevent the proper material balances from being calculated. The high concentration of many contaminants in the early part of storms, referred to as the "first flush", is a common phenomena. It is caused by the rapid mobilization of contaminants attached to fine sediments on impervious surfaces and by the flushing of catchbasins and manholes by the first runoff from a storm.

Obtaining data from the early portion of a runoff event poses difficult logistical problems because of the need for rapid mobilization of sampling crews in response to incoming storms. A substantial budget is required to keep crews in a state of readiness, and many "dry" runs should be anticipated as crews arrive in advance of forecast rainfalls which do not materialize. The need for night and weekend data collection is a further logistical problem. Automatic sampling equipment is a partial solution, but requires extensive set-up and calibration. Also, such equipment is prone to failure and requires frequent checking.

Modelling

Since even a several year period of data collection is inadequate to characterize contaminant loading over a broad range of environmental conditions, the data is

used to calibrate computer models which simulate the loading of contaminants over a more lengthy period. Three main software packages, listed in order of increasing complexity and associated simulation cost, are available for this work: the U.S. Army Corps of Engineers' STORM model, the U.S. Environmental Protection Agency's SWMM III model and the HSPF model. Each model is capable of the long term continuous simulation necessary for the analysis of contaminant loads. The selection of an appropriate model depends on the nature of the problem and the data and budget available for the work.

The STORM model has the advantages of lesser data requirements and simulation costs. It is primarily a loading model used to generate hourly contaminant loads to the receiving water. The SWMM III program is more complex in its characterization of washoff processes and stormwater runoff systems. SWMM III can simulate storage treatment facilities such as retention ponds and receiving water impacts and can be used to determine both the impacts of stormwater runoff and the effectiveness of treatment facilities. HSPF is the most sophisticated and powerful of the models, but has prohibitive data requirements and computing costs. HSPF can simulate loadings, effectiveness of treatment, and receiving water response; since HSPF can also simulate point and rural non-point contaminants, it can be used in the analysis of complex watersheds involving a wide variety of contaminant types and sources.

Once a model has been selected, data and model limitations must be considered. Water quality models are comparatively complex, so experienced personnel are required for their application. Since quality models are significantly less accurate than quantity models, expectations for the final study must reflect these limitations in order to avoid high modelling costs which do not yield the anticipated results. The following points should be kept in mind when designing a water quality modelling study:

a) Initial modelling efforts should be kept as simple as possible. Approximate results may be adequate for decision making, particularly if the costs of more sophisticated modelling are prohibitive. Where detailed modelling is planned from the outset,

coarse screening models should still be used to test whether the detailed simulation will yield useful results.

b) Lumped catchment modelling is often advisable, as the quality of the available runoff and contaminant loading data normally limits the accuracy of the modelling. Detailed discretization usually will not yield further information.

c) Continuous simulation, at least during the screening process is essential. The rate of contaminant build-up and the antecedent moisture conditions have a major effect on loadings. Analysis of contaminant accumulation requires the use of continuous simulation.

d) Selected design storms may be of use in testing treatment facility design and the impact on receiving waters once continuous simulations have been completed. The design storms can be established through frequency analysis of the output from the continuous simulation modelling.

e) Calibration procedures should concentrate first on establishing a good match of runoff volumes and peaks, often by examining a subset of single event simulations. Calibration of contaminant build-up and washoff can then be undertaken in a continuous mode.

f) It is generally advisable to limit contaminant simulation to a few parameters such as TSS, TDS, BOD, TN and coliforms. Other contaminants can often be associated with the loading function for one of these parameters.

The above comments relate to urban runoff modelling for the purpose of estimating contaminant loads. Studies which include modelling of treatment effectiveness may require calibration of settling data. Studies which examine

receiving water response may require assessment of mixing, reaction kinetics, biological uptake and benthic response, sediment/water exchange and contaminant conversion.

7.5 DETENTION PONDS AS TREATMENT FACILITIES

Detention ponds are the most common form of stormwater treatment. Since stormwater quality has only become an issue in recent years, little data is available on their effectiveness, although this is rapidly changing as a result of the National Urban Runoff Program (NURP) in the United States.

In general, treatment ponds remove about 80 - 90% of solids (Chambers, 1978). They are less effective for BOD, TP and TN (with removals of about 50%). They are largely ineffective at removing bacteria. Efficiencies can theoretically be improved by flocculation but the cost of chemicals and labour, the need for special mixing to form flocs, and the need for frequent clean-out usually makes this extra treatment infeasible. Chlorination has been considered for biological control. Factors related to specialized equipment, chlorine storage, reduced effectiveness of chlorine in the presence of solids, trihalomethane formation, and the need to control the chlorine residual in the effluent make chlorination problematic.

Detention ponds designed for quantity control alone are not always suitable for effective water quality control. Special design considerations for quality control ponds include:

a) operation without an operator in attendance,
b) prevention of short-circuiting,
c) minimizing the length in direction of prevailing winds,
d) allowing for sediment accumulation,
e) providing a facility for sediment removal,
f) providing multiple level discharge and skimming capabilities,
g) allowing for make-up water to offset evaporation, and
h) mechanical removal of biomass.

In addition to being effective at removing contaminants, detention facilities must maintain acceptable water quality themselves. Such facilities will not generally be suitable for body-contact recreation. Ponds designed specifically for quality control should be deep, in order to reduce light penetration and minimize temperatures, and should be large to maximize dilution.

7.6 OTHER METHODS OF WATER QUALITY CONTROL

Detention facilities often provide stormwater treatment as a secondary benefit for little additional cost over that required for a runoff control. There are also other treatment methods available for water quality control that can be used either alone or in conjunction with ponds. These include:

a) mechanical devices such as clean-out catchbasins and swirl concentrators,
b) infiltration ponds/galleries,
c) evaporation ponds, and
d) storage/diversion ponds and tanks.

Catchment maintenance practices can be employed to reduce the quantities of contaminants entering the minor system. Catchbasin clean-out and street-sweeping are the most common of these techniques. Recent NURP studies indicate that these practices are not effective against the finer particulates. Such practices provide aesthetic benefit through the removal of coarse materials, but are largely ineffective in reducing contaminant loads.

Devices such as the swirl concentractor have been developed to mitigate combined sewer overflows. These have been discussed in detail by Lager and Smith (1974). Infiltration ponds have been proposed for use in several areas including adjacent to the Dillon reservoir in Colorado. While infiltration ponds have high removal efficiencies, particularly for nutrients, they require suitable soil conditions and large areas of land for their construction. Evaporation ponds

have limited application because of very high land requirements and problems of stagnation and odour. Storage/diversion ponds selectively divert the first flush of stormwater at controlled rates to the sanitary sewer; these can be effective but can also result in significant treatment costs.

7.7 EROSION AND SEDIMENT CONTROL

Post-development control of erosion and sediment is implicit in most developments with stormwater controls. Erosion within the urban catchment is limited since runoff is generally over defined drainage paths, either grassed or impervious. Downstream erosion control can be achieved through constraints on pond outflow rates, although the determination of suitable constraints can be difficult. This is an area where there is need for improvement in the setting of criteria, since maintaining post-development flows to pre-development rates is often inadequate as a result of the much greater runoff volumes created by urban development. Analysis of receiving stream characteristics is necessary to determine the presence of thresholds at which accelerated erosion occurs. It may be necessary to control post-development flows up to a stated frequency to some fraction of their pre-development levels in order to avoid major erosion problems. Since many areas of Alberta have fine-grained soils, this can be a major problem. Armouring critical sections of downstream channels may be the only practical solution in some cases.

Most erosion and sedimentation problems for a development occur during the construction period. A large body of literature has been developed over the last few years addressing this common problem. Control methods that can be used by the developer include buffer strips, filters and temporary sediment traps. If a stormwater facility is part of a development, it is usually rough graded at the start of construction so it may function as a sediment trap.

Even after the development has been completed (in terms of roads, sewers, other utilities and lot grading) there is a considerable period of time during house building and landscaping when the soil is vulnerable to erosion. During this period, the local authority must make sure that proper erosion control practices are used.

8.0 OPERATION AND MAINTENANCE CONSIDERATIONS

There are a number of operation and maintenance activities that must be carried out if a stormwater management system is to adequately serve the public. As stormwater systems are not for primary public health protection (like water supply and sewage collection systems), operational considerations are not a major component of a municipality's annual operation and maintenance budget. However, proper operation and maintenance for any system should be an important part of a municipality's work program regardless of the size of the community. Poor practice in this area will eventually result in great expense and inconvenience to the public and can also be a factor in causing pollution problems in receiving water bodies.

Since the distinction between operation and maintenance activities is sometimes difficult (and most often unnecessary), they are reviewed together in the following discussion.

8.1 MINOR SYSTEM

The minor system components are subject to problems such as erosion, clogging and collapse. Maintenance is required to preserve capacity of the system. Maintenance activities for the minor system can be divided into two types: preventive and corrective (WPCF, 1980).

Preventive maintenance comprises the inspection of the system and the analysis of data related to past complaints and problems. Records of complaints and problems should be documented so that a review of the data will identify key trouble areas. To assist in this, complete records should also be kept of all sewer systems, including:

a) when the sewer was constructed (also, the designer and contractor),
b) type, size and shape of pipe,
c) service area and land use,
d) manholes and catchbasins (location, type and inverts),
e) inspections (date, methods, location and results),
f) complaints (location, nature, date, time, weather), and
g) repairs and replacements.

The physical data should be recorded on "as constructed" drawings, which include the plan and profile of all sewers. As well, overall composite drawings of the entire system are valuable for containing much of the above information. Composite drawings with system information should be in reproducible form and be updated as changes occur.

Preventive maintenance should also include the following activities:

a) cleaning and flushing of streets,
b) sediment removal from catchbasins,
c) supervision of connections and disconnections,
d) steaming of frozen catchbasins, outfalls and culverts,
e) inspection of pipe condition by visual or camera techniques,
f) repair or replacement of damaged pipe, manholes, catch basins and other appurtenances, and
g) review and updating of records.

Corrective maintenance is unscheduled, relating mainly to emergency situations. It pertains to items requiring immediate repair such as fixing pipe breaks, collapses or washouts. Corrective maintenance may also be required to deal with accidents which might allow dangerous materials to enter the sewer system. It is imperative that the first municipal employee at the scene of any accident of this nature make all possible efforts to prevent spilled materials from entering the sewer system (Ontario Ministry of the Environment, 1974). The Alberta Environment Pollution Emergency Response Team (PERT) should be notified of such a situation immediately (phone the toll free PERT number:

1-800-222-6514). Each municipality should have appropriate people on their staff familiar with procedures on how to deal with spills of dangerous materials. Appropriate equipment and materials should be on hand for use in such events.

8.2 DETENTION FACILITIES

8.2.1 General

The municipality is the authority ultimately responsible for the operation and maintenance of stormwater detention facilities. As the municipality may not have responsibility for constructing the facility, it should ensure that detention facilities will be built to an acceptable set of standards. These standards should be established jointly by the municipal Parks and Recreation, Drainage, and Planning Departments to ensure that new facilities will be of value to the public and not be inordinately expensive to maintain (such standards have been discussed in Section 4.4 and an example is outlined in Appendix A.)

Operation of detention facilities is the responsibility of the Drainage Department. As these facilities are usually passive devices, little effort is required. Maintenance responsibilities are usually shared between the Drainage and Parks Departments. In smaller communities these may be a single Public Works Department. The operational and maintenance responsibilities should not be left to developers or homeowners associations. They lack the breadth of knowledge and the material and equipment resources necessary to conduct an adequate or cost effective maintenance program.

There are two types of maintenance activities that must be budgeted when operating detention facilities:

Scheduled Maintenance

Regularly required activities for the aesthetic appearance and recreational utility of the facility should be the responsibility of the Parks and Recreation

Department. These include lawn mowing, shrub trimming, debris removal and ice thickness monitoring. The Drainage Department should be responsible for maintenance related to the water body and control structures. These include water quality monitoring, aquatic weed control and sediment removal.

Unscheduled Maintenance

Unscheduled maintenance will be required from time to time in response to extreme rainfalls or prolonged dry periods. Departmental responsibilities should be similar to that described above for scheduled maintenance. These activities include attendance at detention facilities when they are at flood levels, embankment and shoreline repairs, freeing outlets plugged by debris or ice, low water level control, and the handling of algal blooms. The frequency and cost of these maintenance activities depend on the season, type of pond (wet or dry), the size of the facility, and the objectives of the municipality for the area as part of its landscape and recreational facilities.

8.2.2 Equipment Access

It is imperative that access for maintenance equipment be included in the design of detention facilities (Jones and Jones, 1984). Rapid access to inlet and outlet works, although it will be required infrequently, must be provided. These access points should not be obstructed by fences, landscaping or other works. Access to wet ponds must provide for the eventual need for scrapers or crawler tractors and trucks to remove sediment accumulation on the pond bottom. Sediment removal will also require a means for draining the pond (either built into the outlet structure or by pumping).

8.2.3 Turf and Landscape

Turf and landscape maintenance can be the most significant portion of a detention facility's maintenance budget. It is a regular activity during the spring, summer and early fall. Around wet ponds the homeowners will generally maintain the area adjacent to their property if access and use are solely theirs.

The municipality will be typically responsible for a portion of the land around the perimeter of a wet detention facility (usually between 20% and 50%) that is municipal property and is intended for use by the general public. In a dry pond the banks, the bed, and all other grassed and landscaped areas around the pond will usually be entirely municipal property, and therefore maintenance will be a municipal responsibility. Lot ownership around a dry detention pond usually does not motivate homeowners to share in the responsibility for maintenance of any of the area.

Detention facilities require grass cutting and tree pruning in public areas to the same extent as any other park area. It is advisable to have a minimum of trees in the grassed area around a pond to facilitate operation of ride-on lawnmowers for fast and efficient cutting of grass. Trees should either be spaced far apart or planted in clusters. In dry ponds and around the perimeter of wet ponds, grass cutting may be required as frequently as every one or two weeks during the summer months. In the design of detention facilities, side slopes steeper than 5 to 1 should be avoided as they are difficult to cut with ride-on lawnmowers, particularly when wet conditions exist.

For some dry ponds, aesthetics have not been a high priority during their design. Grass cutting has frequently been ignored for these facilities either intentionally or due to wet conditions caused by poor drainage across the pond bed. This type of pond should not be located near residential areas as it can become an eyesore. Fencing to preclude access should be avoided (safety should be incorporated into the design even if aesthetics are not).

Several Canadian municipalities which maintain detention facilities have indicated that debris and litter is a significant maintenance issue (Michaels et al, 1983). Frequent maintenance visits are required to empty litter containers, pick up wind blown litter, remove floating debris, and check for vandalism. Inlet and outlet structures should be occasionally checked for blockages. Signing and lighting related to the facility should be maintained to minimize safety hazards to the public.

8.2.4 Aquatic Weed and Algae Control

The growth of aquatic weeds in detention ponds is affected by the water depth, turbidity, and the availability of nutrients. Water depth is the major factor for the control of emergent vegetation. When the depth exceeds 1.2 m, emergent vegetation is rarely a problem. This still leaves a potential for weed growth around the perimeter of the pond. Soil sterilization with a chemical such as simazine will restrict growth in this zone for two or more years (IDE, 1983). After this period there are a number of options that can be pursued:

a) Accept the perimeter growth. Many people do not consider emergent vegetation, such as cat-tails, unsightly.

b) Cut and remove the weed growth from either the land or the water. This will be a short-term solution (annual removal will likely be required). The feasibility of this will depend on the condition of the pond side slopes.

c) Drain the pond, remove the weed growth and resterilize the perimeter soil. If chemicals are to be used, the municipality should check with the Pesticide Chemicals Branch of Alberta Environment to ensure that their use is permissible.

d) Lower the water surface for a period of time (such as over the winter) to kill the growth, then re-establish the water level.

The selected method for weed control is a matter of choice, although tolerating the growth is the most economical. The other alternatives involve environmental and aesthetic consequences that must be considered based on local attitudes.

Algal growth will occur in any water body which has an adequate supply of nutrients. These are usually available in detention ponds unless specific maintenance effort is directed at sediment and nutrient removal. Prolonged

warm weather encourages algal growth. Algal blooms are most likely to occur in areas of the lake adjacent to the inlets, and are most effectively treated by chemical application; copper sulphate has been used extensively in the past with good results. Other chemicals such as Simazine and Diquat will also control such plants as coontail, duckweed, pond weeds, water milfoil and certain algal species (The Sibbald Group, 1978).

The timing of the application of these chemical products is very important. For example, Simazine is most effective if applied immediately following the spring thaw and prior to the summer rains. In lakes known to have weed growth problems later in the year, this preventive measure may provide the most effective form of weed control. In contrast, Diquat is a contact weed killer and must be applied when weeds are actively growing. The possibility of water contact by the public is to be prevented for at least 24 hours after application. Weeds will die one to two weeks after application.

Most chemical products require a permit for use from the Pesticide Chemicals Branch of Alberta Environment, and the personnel applying them must be licensed. Those chemicals mentioned were approved for use in 1985. As new products enter the market, the recommended chemicals and applications will likely change. Municipalities should review potentially useful chemicals and procedures at regular intervals.

8.2.5 Mosquito Control

The effect of detention ponds on mosquito populations is a subject which has rarely been addressed in the literature. Only a few Canadian municipalities practice mosquito control on detention ponds (Michaels et al, 1983). Some jurisdictions have used chemical sprays or pellets applied to the pond surface to control mosquitoes. In Alberta, approval must be received from the Pesticide Chemicals Branch of Alberta Environment prior to using any chemicals for mosquito control. In Winnipeg, fish have selectively been used for mosquito control. Agitating the pond surface by circulating water or using aeration equipment has also been used to reduce mosquito populations. Grass cutting may

provide a supplementary benefit by somewhat reducing adult mosquito populations near a pond.

8.2.6 Aeration and Circulation

Aeration and circulation equipment can be used to make the pond environment less conducive to the production of algae and mosquitoes. Some researchers, however, have found that aeration has enhanced algal production where oxygen, not sunlight or nutrients, is the limiting factor. Aerating the lakes generally aids the decomposition of algae and other dead biomass. This helps alleviate odour but is a cure rather than a preventative measure. A better approach is to prevent excessive growth of algae by reducing nutrient inflow by trapping sediment upstream of the pond inlets. If the growth of weeds and algae is controlled, recycling of nutrients through decomposition is also reduced. In addition, oxygen levels will not be depleted, and odour problems can be avoided.

A wet pond should be designed so that "dead bay" areas in the pond are avoided. The strategic location of inlets and outlets may generate sufficient flushing action to achieve this objective. Aeration and circulation equipment should not be mandatory in the design of detention facilities. Their value is for retrofitting to solve problems which have occurred, or to improve aesthetics.

8.2.7 Makeup Water

Makeup water can also assist in the control of mosquitoes and algae if it is introduced in a turbulent fashion at the right locations. In addition, if the makeup water is good it assists the water quality situation by dilution. The most appropriate source for good quality makeup water is the municipal system (Hodgson and Stalker, 1983). Due to the high cost of this water, it should be utilized only for water quality control and not water level control. A lake with the appropriate side slopes and shoreline treatment will not show unsightly "mudflats" when the water falls below normal water level. In most of Alberta,

groundwater is not an appropriate source of makeup water. Generally, groundwater is a scarce resource and makeup water for a detention pond is not a high priority use.

8.2.8 Winter Activities

Maintenance activities for detention facilities are reduced and are of a dramatically different nature during the winter. Wet ponds require the most maintenance attention during the winter months. The most active periods are during freeze-up in early winter and during the spring thaw.

Many stormwater detention ponds are used for ice skating during winter. If the municipality chooses to support this recreational activity for the community, they can become involved in the establishment and maintenance of skating facilities. Otherwise, the municipal authority should be quite clear that people use the lakes at their own risk and leave it to them to clear skating areas.

At the beginning of winter, some municipalities place announcements in the local newspapers or on radio to alert the residents that it may be unsafe to use the lake surface during winter. Others present a firmer position, and indicate by the placement of permanent "thin ice" signs around the pond that use of the lake surface is at the public's own risk (an implicit responsibility). Considering the possibility of unsafe ice conditions elsewhere on the pond's ice surface (the area not being maintained by the municipality), the latter policy has considerable merit.

The selection of skating areas is primarily determined by the accessibility of the sites to maintenance equipment and user facilities for the public. On some ponds the entire surface area may be kept free of snow to allow skating. On others, one or more local cleared areas can be used. Where isolated areas are to be used, these should be located in quiet water areas, away from any inlet or outlet works in order to minimize the possibility of localized thermal erosion of the ice. Areas that have this problem should be barricaded, fenced, or have signs to signal the danger.

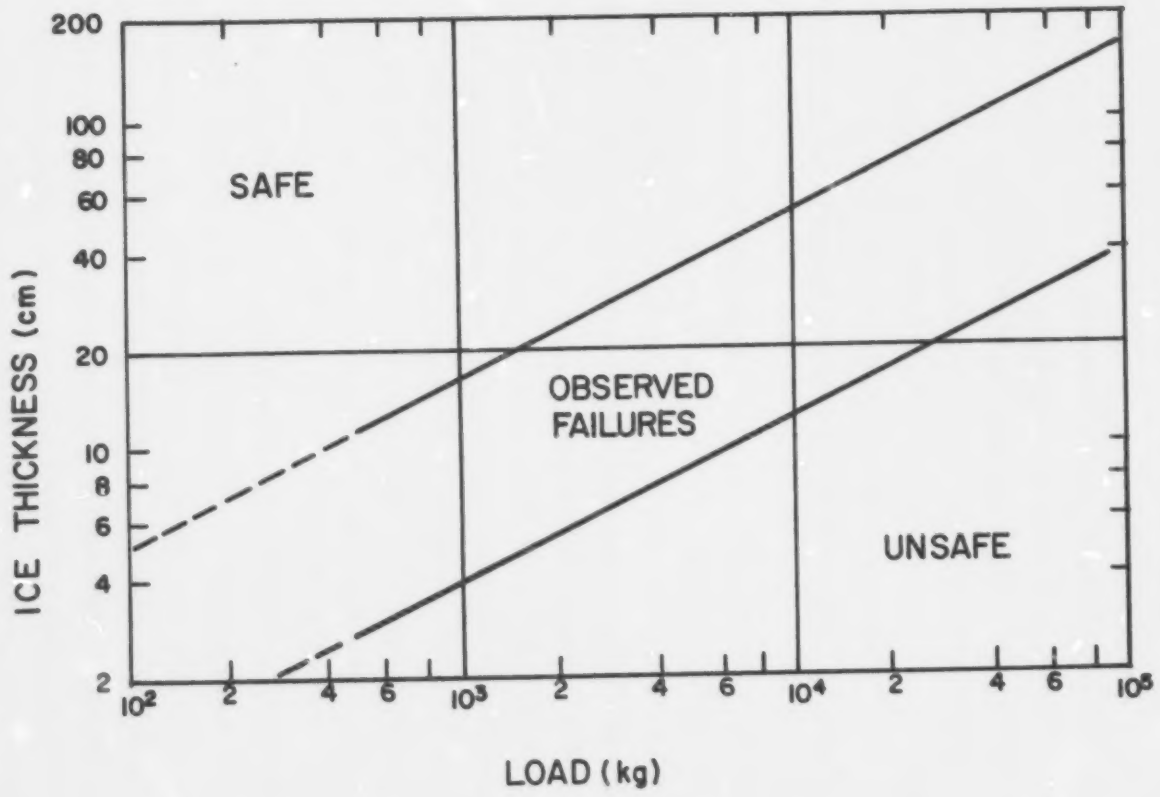
Natural ice grows from the water/ice interface downwards. As the ice thickness increases, the rate of growth decreases (it becomes more difficult for the heat in the warm water to escape to the cold atmosphere). On the other hand, flooding the ice surface will result in the establishment of an ice surface more quickly. This is mainly because the primary freezing surface is in direct contact with the colder atmosphere. In either case, it is important to keep the ice surface free of snow during cold weather as the insulating effects of snow greatly reduce the rate of ice growth.

An ice surface is not safe to carry a load until it has reached a sufficient thickness (Figure 8.1). Typically, holes are drilled at several locations to check the area for ice thickness and uniformity. Drilling is done frequently until it is possible to support snow clearing equipment. Once this "load test" has been completed there is no need for further flooding (other than for ice surface restoration) or drilling as long as the weather remains cold.

In the spring, an ice surface deteriorates and usually floods with meltwater prior to the ice becoming unsafe to walk on. This condition hopefully discourages use of the lake during the winter/spring transition. As the spring melt progresses, children are sometimes tempted to walk or bicycle on the lake surface when it is unsafe to do so. In the interests of public safety, maintenance of a skateable ice surface should cease at the earliest signs of a warming trend. Frequent site visits during thawing periods (particularly, after school and on weekends) should be scheduled to discourage activities on the pond.

8.2.9 Sediment Control

Sediment control is one of the most important activities in maintaining the water quality of a stormwater detention facility. It is also one of the activities that is the easiest to ignore (in the short term). Stormwater detention facilities will serve as a sink for most materials that impair water quality; the facilities will improve the quality of the discharge because the detention period will allow the settling of suspended material and parameters which are bound to it. Some parameters may also be reduced by detention through biological activity,



ICE FAILURE / LOAD OBSERVATIONS

FIGURE 8.1

volatization or die-off. However, the continued discharge of sediment to the facility will ultimately degrade the pond's water quality to the point where it may be unacceptable to the public from either an aesthetic viewpoint (e.g. algal blooms) or a health viewpoint.

Monitoring of certain water quality parameters is required to ensure that the detention facility does not deteriorate beyond acceptable standards (see Section 8.2.10). Sediment control is the best means to prevent unacceptable deterioriation from occurring. Although sediment is only one water quality parameter which can have adverse impacts on the environment, it is the most easily observed. As many other contaminants are associated with sediment, control of sediment will in effect control most of these substances.

In the post-development period, the first opportunity for sediment control comes at the catch basin sumps. These sumps are used in many cities in Alberta. Catch basins can be inconvenient to maintain as they require the servicing of a large number of minor facilities over the serviced area. Since present sewer systems are designed with self scouring velocities, it can be more convenient to install a smaller number of large sediment traps in the system. These can be located near the inlets to the storm ponds. Sediment removal structures located upstream of a pond can be reliably serviced on a frequent basis by eductor trucks. Servicing these facilities is required after each significant rainfall event and during the spring snowmelt runoff period.

Most stormwater systems convey sediment into the detention facility. The City of Winnipeg indicates that the sediment accumulation in their facilities is in the order of 2.5 mm per year (Michaels et al, 1983). Winnipeg monitors the bottom topography of its lakes every five years to determine the need for widespread sediment removal. The original philosophy regarding sedimentation was that after a prolonged period of development, dredging or draining and excavation would have to be undertaken to restore the lake bed to the originally designed elevation. If necessary, this would occur after full urbanization of the drainage basin has occurred and sedimentation had dropped to negligible rates. After an urban watershed has been fully developed it was expected that there would be

little need to again consider dredging for many years (in the order of 20 to 50 years). Experiences in the Prairies to date indicate that few ponds, if any, have required any sediment removal. The need for ongoing sediment control and removal is related to water quality control and not to preventing a loss of storage volume.

Sediment control problems with dry ponds (equipped with a bypass to convey runoff from the more common runoff events) relate to sediment removal from a sump which should be incorporated into the design of the bypass structure. When the pond is inundated during a less frequent event there may be some need to remove silt deposits on the grass near the inlet. For dry ponds designed with the through flow concept, the silt depositions will be more significant and more frequent.

8.2.10 Monitoring

It is economically impractical to treat detention pond water to comply with quality standards established for contact recreational activities. As a result, it is necessary to address the water quality of a wet detention facility with the intent of accommodating secondary and tertiary recreational activities (if primary contact recreation is allowed, more stringent criteria are necessary). Secondary recreational activities relate to non-contact water based activities such as boating, skating and cross-country skiing. Tertiary recreational activities relate more to aesthetic considerations and include cycling, hiking, picnicing and other activities which would benefit from the visual enhancement that the water body provides to the surrounding area.

Where a wet pond is intended to be a multi-use facility (i.e. allowing secondary and tertiary recreational activities), the quality of water in the pond should compare with the Surface Water Quality Objectives established by Alberta Environment (1977). Except for a short period following a runoff event, the quality of water in a pond should be expected to meet these objectives. Where concentrations exceed these objectives significantly, particularly coliform counts, the source of such contaminant loads should be found and eliminated.

The more important parameters are summarized in Table 8.1 (refer to ASWQO for a more detailed statement of the guidelines).

Michaels et al (1983) indicated that monitoring practices for stormwater detention ponds vary widely. Of the cities who monitor water quality, the sampling frequency ranged from weekly to twice a year. A monthly monitoring program closely represents the median of monitoring practice.

TABLE 8.1

RECOMMENDED QUALITY LEVELS FOR DETENTION PONDS

	Parameter	Recommended Level
o	Total Coliforms	2,400 per 100 ml at any time, and 1,000 per 100 ml (geometric mean) when five or more samples taken in a 30 day period.
o	Fecal Coliforms	200 per 100 ml (geometric mean) when five or more samples taken in a 30 day period.
o	Dissolved Oxygen	Greater than 5 mg/L.
o	Biochemical Oxygen Demand (BOD_5)	Not to exceed a level which would create dissolved oxygen content of less than 5 mg/L.
o	Turbidity	Less than 25 Jackson units above natural (receiving stream) level.
o	Colour	Less than 30 colour units above natural (receiving stream) level.
o	Odour	The cold (20o C) threshold odour number not to exceed eight.
o	Nitrogen (total inorganic and organic)	Less than 1.0 mg/L.
o	Phosphorus (total inorganic and organic, as PO_4)	Less than 0.15 mg/L.

Source: Alberta Surface Water Quality Objectives (Alberta Environmental, 1977).

As it is inevitable that the public will have some contact with the water, monitoring of bacteria is important to determine the possibility of risk to public health. Control of BOD and dissolved oxygen levels ensures that the water body remains aerobic and lessens the likelihood of odour problems. Turbidity, colour and odour all relate to aesthetic considerations. Monitoring nutrients can aid in predicting nuisance problems associated with algal blooms (SAEL, 1981), which also present aesthetic concerns.

Monitoring of toxic chemicals (such as lead, zinc, copper, mercury, etc.) may be done on a less frequent basis. Quarterly monitoring is commensurate with the above-mentioned recreational usage. Although stocking a pond with fish may be desirable from a recreational perspective, it is not desirable from a public health perspective. The contaminants (chemical and biological) present in stormwater are likely to accumulate in the fish flesh and can be transmitted to humans. Where stormwater ponds are stocked, both fishing and the consumption of fish should be forbidden, and the monitoring of toxic chemicals in both the pond water and the pond sediment should be more frequent.

8.2.11 Operation and Maintenance Costs

Costs for the operation and maintenance of detention facilities have not been well documented. Michaels et al (1983) indicated that only three of all the cities they surveyed provided costs for the maintenance of wet detention facilities. No data was available for the maintenance costs of dry ponds. For dry ponds, the operation and maintenance costs are expected to be less as they relate to only inspection, debris control, and turf and landscape maintenance. Reasonable estimates for these costs could be made by an experienced parks and recreation department.

The operation and maintenance of wet detention facilities are usually based on the surface area of the pond. Michaels et al (1983) described a study where the maintenance activities for a hypothetical wet pond were outlined and costed. The costs were estimated to be about $2,000/ha of water surface (1971 dollars). This is about $5,300/ha in current (1985) dollars. For the cities who documented

costs, the mean was $1,500/ha (1971 dollars), representing a current cost of about $4,000/ha.

8.3 PUMPING FACILITIES

Pumping facilities are infrequently used in stormwater management systems, although they are sometimes necessary in conjunction with retention storage in areas of very low relief. Pumping stations are a major concern in stormwater management systems because they are infrequently utilized and the consequences of their failure can be severe in terms of the related flooding damage that can occur. Most of the activities and design considerations in the sanitary sewer maintenance manuals (Ontario Ministry of the Environment, 1974; WPCF, 1980) are applicable to pumping facilities for stormwater management systems.

REFERENCES

BIBLIOGRAPHY

Ahmad, M., 1980a. Methodology for 'Lumped' SWMM Modelling. SWMM Users Group Meeting. U.S. Environomental Proctection Agency.

Ahmad, M., 1980b. Stormwater Modelling Applications, City of Edmonton. SWMM Users Group Meeting. U. S. Environmental Protection Agency.

Alberta Environment, 1976. Objectives for Stormwater Management. Standards and Approvals Division. Edmonton, Alberta.

Alberta Environment, 19__. Stormwater Management Procedures and Guidelines for Runoff Control in New Developments. Edmonton, Alberta.

Alberta Environment, 1977. Surface Water Quality Objectives. Water Quality Branch. Edmonton, Alberta.

Alberta Environment, 1978. Recommended Standards for Water Supply and Sewage, Wastewater Treatment, Sewage Treatment, Storm Drainage. Edmonton, Alberta.

Albery, Pullerits, Dickson and Associates Limited, 1976. Practices, Policies and Technology of Storm and Combined Sewers in Foreign Countries. Research Report No. 45. Canada - Ontario Agreement on Great Lakes Water Quality.

American Iron and Steel Institute, 1980. Modern Sewer Design, Canadian Edition. Washington, D.C.

American Iron and Steel Institute, 1984. Handbook of Steel Drainage and Highway Construction Products, Canadian Edition. Washington, D.C.

American Public Works Association, 1981. Urban Stormwater Management Special Report No. 49. Chicago, Illinois.

Amy, G., R. Pitt, R. Singh, W. L. Bradford, and M. B. LaGraff, 1974. Water Quality Management Planning for Urban Runoff. U.S. Environmental Protection Agency.

Anderson, J. J., 1975. Economics of Stormwater Detention. APWA/Urban Drainage Workshop.

Andrew Brodie Associates Inc., 19__. Urban Drainage Implementation Guidelines, Major and Minor Drainage Systems. Technical report for Technical Sub-Committee on Major-Minor Guidelines.

AESL, 1982. Evaluation of the Hydrologic Model HYMO for Application in Alberta. A technical report prepared for Alberta Environment, Technical Services Division. Edmonton, Alberta.

AES, 1982. Rainfall Intensity-Duration-Frequency Graphs for Alberta Stations. Downsview, Ontario.

APWA Research Foundation, 1981. Urban Stormwater Management. American Public Works Association.

ASCE, 1977. Guide for Collection Analysis and Use of Urban Stormwater Data. Proceedings, Urban Runoff, Stormwater Collection Analysis & Use. Engineering Foundation, U.S. Geological Survey, and ASCE Urban Water Resources Research Council.

ASCE, 1975. Urban Water Resources Research Council Urban Runoff Quantity & Quality. Proceedings, Environmental Impact, Flood Control, Storm Drainage. Engineering Foundation.

ASCE, 1970. Design and Construction of Sanitary and Storm Sewers. ASCE 37. WPCF Manual of Practic No. 9. New York, New York.

Babroski, G. J. and S. R. Goswani, 1979. Urban Development and Flood Control Design. Public Works.

Bandyopadhyay, M., 1972. Synthetic Storm Pattern and Run-off For Gauhati, India. ASCE Journal of the Hydraulics Division. Vol. 98, No. HY5.

Barnwell, T. O. Jr. (editor), 1982. Proceedings of Stormwater and Water Quality Management Modelling Users Group Meeting. U.S. Environmental Protection Agency. Washington, D.C.

Bartlett, R. E., 1976. Surface Water Sewerage. Halstead Press. Toronto, Ontario.

Beckett, Jackson, Raeder Inc., 1981. Soil Erosion and Sediment Control. Environmental Design Press.

Blake-Coleman, W. (editor), 1984. Urban Effects on Water Quality and Quantity. Authority of the State of Illinois, Department of Energy and Natural Resources.

Booy, C., K. W. Lathem and W. E. Watt, 1980. Flood Hydrology in Canadian Summary of Current Practice. Associate Committee on Hydrology.

Burton, K. R., 1980. Stormwater Detention Basin Sizing. ASCE Journal of the Hydraulics Division. Vol. 106, No. HY3.

Chow, V. T., 1959. Open-Channel Hydraulics. McGraw-Hill.

Chambers, G. M. and C. H. Tottle, 1978. Evaluation of Stormwater Impoundments in Winnipeg. Waterworks, Waste and Disposal Division. City of Winnipeg, Manitoba.

City of Burlington, 1977. Storm Drainage Criteria Manual. A manual prepared by M.M. Dillon Ltd.

City of Calgary, 1980. A Policy on Stormwater Retention Lakes for the Calgary North Area. Interdepartmental Task Force. Calgary, Alberta.

City of Edmonton, 1979. Servicing Standards Manual, Fourth Edition. Land Development Coordination Branch. Edmonton, Alberta.

City of Edmonton, 1980. Policy on Stormwater Lakes (draft). Edmonton Parks and Recreation, Planning and Development Branch, Research and Planning Section. Edmonton, Alberta.

City of Edmonton, 1982. Draft Servicing Standards. Edmonton, Alberta.

City of St. Paul, 1976. A Comprehensive Policy for the Ponding of Stormwater. Department of Public Works. St. Paul, Minnesota.

City of Winnipeg, 1974. Stormwater Management by Use of Impoundments. Works and Operations Department. Winnipeg, Manitoba.

Cole, R.H., R. E. Frederick, R. P. Healy and R. G. Rolan, 1984. Preliminary Findings of the Priority Pollutant Monitoring Program. Journal of the Water Pollution Control Federation. Vol. 57, No. 7.

County of Strathcona, 1983. Stormwater Lake Guidelines. Recreation and Parks, Engineering. Sherwood Park, Alberta.

Dance, K. and D. Cressman, 1981. Critical Review of the Cost-Effectiveness of Various Erosion and Sediment Control Techniques. Ecologists Limited.

Dean, J. D. and W. M. Snyder, 1977. Temporally and Areally Distributed Rainfall. ASCE Journal of the Irrigation and Drainage Division. Vol. 103, No. IR2.

District of Coquitlam, 1981. Subdivision Control Bylaw No. 1023. Coquitlam, British Columbia.

Edmonton Regional Planning Commission, 1980. Stormwater Impoundment Area Policy. Edmonton, Alberta.

Environment Canada, 1978a. Microbiological Characteristics of Urban Stormwater Runoffs in Central Ontario. Research Report No. 87. Canada-Ontario Agreement on Great Lakes Water Quality.

Environment Canada, 1978b. Stormwater Management Model Workshop, Conference Proceedings. Report No. 4.

Environment Canada, 1977a. Modern Concepts in Urban Drainage. Conference Proceedings. Report No. 5.

Environment Canada, 1977b. Manual of Practice on Urban Drainage. Urban Drainage Subcommittee.

Ferrara, R. A., 1982. Stormwater Detention and Nonpoint Source Pollution Control. Proceedings of 1982 International Symposium on Urban Hydrology, Hydraulics and Sediment Control. University of Kentucky.

Ferrara, R. A. and P. Witkowski, 1983. Stormwater Quality Characteristics in Detention Basins. ASCE Journal of the Environmental Engineering Division. Vol. 109, No. EE2.

Field, R., 1980. EPA Research in Urban Stormwater Pollution Control. ASCE Journal of the Hydraulics Division. Vol.106, No. HY5.

Field, R., H. Masters and M. Singer, 1982. An Overview of Porous Pavement Research. Water Resources Bulletin. Vol. 18, No. 2.

Finnemore, E. J., 1982. Stormwater Control: Best Management Practices. ASCE Journal of the Environmental Engineering Division. Vol. 108, No. EE5.

Frederick, R.H., 1978. Interstorm Relations in Pacific Northwest. ASCE Journal of the Hydraulics Division. Vol. 104, No. HY12.

Ganczarczyk, S. S., 1976. Storage for Stormwater Quality Control. SWMM Users Group Meeting. U. S. Environmental Protection Agency. Washington, D.C.

Gietz, R. J. 1981. Stormwater Runoff Treatment by Impoundment. Barrhaven Pilot Study.

Gill, D. G. and P. F. Kelly, 1978. Stormwater Management by the Use of Impoundments and Their Application in Western Canada. American Public Works Association, Calgary Regional Conference.

Gloyna, E. F., J. F. Malina and E. M. Davis (editors), 1976. Ponds as a Wastewater Treatment Alternative. Water Resources Symposium No. 9. Center for Research in Water Resources, College of Engineering, The University of Texas at Austin.

Goff, J. D. and F. E. Goldman, 1981. Urban Recreational Lakes Servicing as Stormwater Retention Basins and Effluent Disposal. Eighth International Symposium on Urban Hydrology, Hydraulics and Sediment Control. University of Kentucky.

Gold, L.W., 1971. Use of Ice Covers for Transportation. Canadian Geotechnical Journal. Vol. 8.

Gore and Storrie Limited, 1979. Stormwater Management for Nepean, Merivale Area. Research Report No. 89. Canada-Ontario Agreement on Great Lakes Water Quality.

Gore and Storrie Limited. 1978. Review of Canadian Municipal Urban Drainage Policies and Practices. Research Report No. 82. Canada-Ontario Agreement on Great Lakes Water Quality.

Gray, D. M. (editor), 1970. Handbook on the Principles of Hydrology. National Research Council of Canada. Ottawa, Ontario.

GVS and DD, 1975. A Comparison of Three Runoff Simulation Models. Vancouver, British Columbia.

GVS and DD, 1976. The Rational Method of Peak Storm Flow Determination. Vancouver, British Columbia.

GVS and DD, 1977a. Review of Methods for Computing Storm Runoff from Urban Areas -History and State of the Art. Vancouver, British Columbia.

GVS and DD, 1977b. Report on a General Hydrological Study with Emphasis on Data Base and on General Principles Design Criteria and Aids. Vancouver, British Columbia.

Grizzard, T. J., B. L. Weand and C. W. Randall, 1982. An Evaluation of Stormwater Ponds for the Control of Urban Runoff Pollution. Proceedings of the First International Seminar - Urban Drainage Systems. Southampton, England.

Haan, C. T., 1979. Statistical Methods in Hydrology. The Iowa State University Press. Ames, Iowa.

Haan, C. T. (editor), 1978. Proceedings International Symposium on Urban Stormwater Management.

Haan, C. T., 1977. Urban Hydrology Frequency Analysis. University of Kentucky.

Henry, J. G., 1976. The Effect of Storage on Storm and Combined Sewers.

Hey, D.L. and G.C. Schaefer, 1984. An Evaluation of the Water Quality Effects of Detention Storage and Source Control. Conference Proceedings: Urban Effects on Water Quantity and Quality. Illinois Department of Natural Resources.

Hill, D. W. and Wright, K. R., 1971. Urban Water Resources Management. American Society of Civil Engineers.

Hodgson, J. E. and B. P. Johnson, 1985. Stormwater Management Trends in Western Canada. B.C. Water and Waste Association Annual Meeting. Penticton, British Columbia.

Hodgson, J. E. and M. J. Stalker, 1983. The Operation and Maintenance of Stormwater Detention Facilities. Western Canada Water and Sewage Conference Annual Meeting. Edmonton, Alberta.

Hodgson, J. E. and D. Wilson, 1980. Spreading Transition University of Alberta. Edmonton, Alberta.

Hogg, W. D., 1980. Time Distribution of Short Duration Storm Rainfall in Canada. Proceedings, Canadian Hydrology Symposium: 80, Toronto. National Research Council of Canada. Ottawa, Ontario.

Hogg, W. D., 1982. Distribution of Rainfall With Time: Design Considerations. Presented to American Geophysical Union. Chapman Conference on Rainfall Rates. Urbana, Illinois.

Huber, W. C., J. P. Heaney, S. J. Nix, R. E. Dickinson, and D. J. Polmann, 1983. Stormwater Management Model User's Manual, Version III.

Huber, W. C., May 1975. Modelling for Stormwater Strategies. AWWA Reporter.

Huber, W. C., J. P. Heimey and H. C. Torno, 1977. Computer Impact on Environmental Runoff and Wastewater Studies. ASCE.

Huber, W. C., 1974. Use of Modelling for Planning Stormwater Strategies. Public Works Congress.

Hurst, W. D., 19__. The Management of Urban Drainage - A Brief Consideration of the History, Background and Future Prospects of the Art. American Public Works Association. Chicago, Illinois.

Hydraulics Division ASCE, 1978. Proceedings of the 26th Annual Hydraulic Specialty Conference.

Hydrocomp International, 1973. Computer Simulation for Design Criteria for Urban Flow Storage Systems. Stormwater Management.

IDEL, 1983. Stormwater Management (SWM) Lakes - Weed Control Guidelines. Unpublished.

Illinois State Water Survey, 1974. The Illinois Urban Drainage Area Simulator, ILLUDAS. Bulletin 58. Urbana, Illinois.

J. F. MacLaren Ltd., 1980. Brucewood Urban Test Catchment. Research Report No. 100. Canada-Ontario Agreement on Great Lakes Water Quality.

J. F. MacLaren Ltd., 1978. Report on Design Storm Selection. City of Winnipeg.

J. F. MacLaren Ltd., 1974. Drainage Criteria Manual for the City of Winnipeg. Willowdale, Ontario.

J. F. MacLaren Ltd., 1980. Stormwater Management Techniques System-Demonstration, City of St. Thomas. Environment Canada/CMHC.

J. F. MacLaren Ltd., 1979. Edmonton Rainfall Analysis. A Comparison of Historical and Theoretical Design Storms. A report to the City of Edmonton. Edmonton, Alberta.

J. F. MacLaren Ltd., 1975. Review of Canadian Design Practice and Comparison of Urban Hydrologic Models.

James, W. and R. H. McCuen, 1983. Emergency Computer Techniques in Stormwater/Flood Management. ASCE Proceedings.

James, W. and M. A. Robinson. Terms of Reference Common to Computer-Based Stormwater Studies.

Johnson, R. L. and R. A. Putt, 1977. Stormwater Retention and Detention.

Jones, J. E. and D. E. Jones, 1984. Essential Urban Detention Ponding Considerations. ASCE Journal of Water Resources Planning and Management. Vol. 110, No. 4.

Kamedulski, G. E., and R. H. McCuen, 1978. Comparison of Stormwater Detention Policies. ASCE Specialty Conference of the Hydraulics Division.

Kassem, A.M. and Wisner, P.E., 1980. Validity of WRE-TRANSPORT. IMPSWM Progress Report No. 12. University of Ottawa.

Keifer, D. J. and H. H. Chu, 1957. Synthetic Storm Pattern for Drainage Design. ASCE Journal of the Hydraulics Division. Vol. 83, No. HY4.

Khanal, N. N., 1975. Hydrologic Aspects of On-Site Retention Systems for Urban Storm Runoff. Cental and Southern Florida Flood Control District.

Kibler, D. F. (editor), 1982. Urban Stormwater Hydrology. Water Resources Monograph #7 American Geophysical Union.

King, C.H. and G. M. Ward, 1973. The Perimeter Canal - A New Approach to Discharge Urban Water Into a Coastal Mangrove Area. Proceedings from Specialty Conference on Agricultural and Urban Considerations in Irrigation and Drainage. Fort Collins, Colorado.

Kronis, H., 1978. Characterization and Treatment of Snowmelt Runoff from an Urban Catchment. Research Publication No. 73. Ontario Ministry of the Environment.

Lager, J. A. and W. C. Smith, 1974. Urban Stormwater Management and Technology, As Assessment. Program Element No. 1BB034. Contract No. 68-03-0179 U.S. Environmental Protection Agency.

Lai, C., 1980. Sources of Computer Programs in Hydraulics. ASCE Journal of the Hydraulics Division. Vol. 106, No. HY5.

Linsley, R. K., M. A. Kohler and J. L. H. Paulhus, 1977. Urban Runoff Model JCL and Procedures.

Linsley, R. K., M. A. Kohler and J. L. H. Paulhus, 1975. Hydrology for Engineers. McGraw-Hill.

Marsalek, J. and W. E. Watt, 1984. Design Storms for Urban Drainage Design. Canadian Journal of Civil Engineering. Vol. 11, No. 4.

Marsalek, J., 1979. Malvern Urban Test Catchment. Volume 2. Research Report No. 95. Canada-Ontario Agreement on Great Lakes Water Quality.

Marsalek, J., 1978. Instrumentation of Field Studies of Urban Runoff. Environment Canada.

Marsalek, J., 1977. Malvern Urban Test Catchment. Volume 1. Research Report No. 57. Canada-Ontario Agreement on Great Lakes Water Quality.

Marsalek, S., 1975. Urban Runoff Modelling. APWA.

Marshall Macklin Monaghan Limited, 1982. Watershed Plan - Northwest Annexation Area, Edmonton. Edmonton, Alberta.

McCuen, R. H., 1979. Downstream Effects of Stormwater Management Basins. ASCE Journal of the Hydralics Division. Vol. 105, No. HY11.

Mein, R. G., 1975. Analysis of Detention Basin Systems. SWMM Users Group Meeting. U. S. Environmental Protection Agency. Washington, D.C.

Metropolitan Toronto and Region Conservation Authority, 1977. Stormwater Management Seminar.

Michaels, S., E. McBean and G. Mulamoottil, 1983. The Canadian Stormwater Impoundment Experience. A research project for School of Urban and Regional Planning. University of Waterloo.

Mills, W. G., 1977. Water Quality of Urban Stormwater Runoff in the Borough of East York. Research Report No. 66. Canada-Ontario Agreement on Great Lakes Water Quality.

Ministry of the Environment, 1980. Practical Implementation of Stormwater Management Technique. Ministry of the Environment and Municipal Engineers Association.

Modern Concepts in Urban Drainage, 1978. March 28 - 30, 1977. Proceedings No. 5.

Mukherjee, D., D. Keliar, P. Wisner and A. Brodie, 1983. Stormwater Management in the Town of Markham. Markham, Ontario.

Murrey, M. P. H., 1977. Storage for Storm Water Quality Control, Meadowvale Test Site Study. Research Report No. 63. Canada-Ontario Agreement on Great Lakes Water Quality.

Municipal Engineers Association of Ontario, 1977. Municipal Works Design Manual, 1st Edition.

Nicks, A. D. and F. A. Igo, 1980. A Depth-Area-Duration Model of Storm Rainfall in the Southern Great Plains. Water Resources Research. Vol. 16, No. 5.

Oliver, L. J. and S. G. Grigoropoulos, 1981. Control of Storm - Generated Pollution Using a Small Urban Lake. Journal of the WPCF. Vol. 53, No. 5.

Ontario Ministry of the Environment, and Environment Canada, 1977. Proceedings from Conference on Modern Concepts for Urban Drainage Manual. Toronto, Ontario.

Ontario Ministry of the Environment and Municipal Engineers Association - Ontario, 1974. Sewer Maintenance Manual.

Ontario Ministry of Natural Resources, 1981. Stormwater Management Guidelines, Central Region. Richmond Hill, Ontario.

Overton, D. E. and M. E. Meadows, 1976. Modern Practices for Master Drainage Plans and Verification/Review Agencies. Stormwater Modelling Textbook. Academic Press.

Pagan, A. R., 1974. Calculating Outflow from Small Reservoirs. Water and Sewage Works.

Patry, G. G. and M. A. Marino, 1984. Two Stage Urban Runoff Forecast Model. ASCE.

Patry, G., L. Raymond, and G. Mandin, 1979. Interactive Mini-Computer Version ILLUDAS Model. SWMM Users Group Meeting. U. S. Environmental Protection Agency. Washington, D.C.

Paul Theil Associates Ltd., 1976. Zero Increase in Stormwater Runoff - A New Concept in Storm Water Management. A technical report prepared for the Housing and Urban Development Association of Canada (HUDAC). Toronto, Ontario.

Poertner, H. G., 1974. Practices in Detention of Urban Stormwater Runoff, Special Report No. 43. American Public Works Association.

Poertner, H. G., 1973. Better Storm Drainage Facilities - At Lower Cost. Civil Engineering - ASCE. October, 1973.

Poertner, H.G., 1976. Urban Stormwater Detention and Flow Attenuation. Public Works Magazine. August, 1976.

Ports, M. A., 1975. Sediment and Erosion Control Design Criteria. APWA Reporter. May, 1975.

Proctor and Redfern Limited and J. F. McLaren Ltd., 1976. First Canadian Stormwater Management Model Workshop. Environment Canada.

Proctor and Redfern Limited and J. F. McLaren Ltd., 1976. Storm Water Management Model Study, Vols. 1, 2 and 3. Technical Background. Research Report Nos. 47, 48 and 62. Canada-Ontario Agreement on Great Lakes Water Quality.

Proctor and Redfern Limited, 1979. Application of Computer Models on an Urban Drainage Relief Study. A paper submitted for 6th International Symposium on Urban Storm Runoff. Kentucky, USA.

RTAC, 1982. Drainage Manual, Vol. 1. Roads and Transportation Association of Canada. Ottawa, Ontario.

Russel, S. O., B. F. I. Kenning and G. J. Sunnell, 1979. Estimating Design Flows for Urban Drainage. ASCE Journal of the Hydraulics Division. Vol. 105, No. HY1.

SAEL, 1981. General Guidelines for Stormwater Ponds. A report to the County of Strathcona. Edmonton, Alberta.

Scherger, D. A. and J. A. Davis, 1982. Control of Stormwater Runoff Pollutant Loads by a Wetland and Detention Basin. Proceedings of 1982 International Symposium on Urban Hydrology, Hydraulics and Sediment Control. University of Kentucky.

Seto, P., 1983. Ontario Urban Drainage Management Program. Ministry of the Environment.

Sibbald Group, 1978. Weed Control Alternatives for Stormwater Retention Pond Located at Leduc, Alberta. A report to Stanley Associates Engineering Ltd. Edmonton, Alberta.

Simanton, J. R., and H. B. Osborn, 1980. Reciprocal - Distance Estimate of Point Rainfall. ASCE Journal of the Hydraulics Division. Vol. 107, No. HY7.

Storr, D., 1967. A Frequency Analysis of Maximum Two-Day and Three-Day Rainfalls in Saskatchewan, Alberta and Northeastern British Columbia. Meteorological Branch, Department of Transport. Toronto, Ontario.

Sullivan, Richard H. et al, 1978. Evaluation of the Magnitude and Significance of Pollution from Urban Stormwater Runoff in Ontario.

Switzer, S., 1981. Implementation Aspects of Stormwater Management. University of Ottawa.

Theill, P. E. and A. M. Canderas, 1978. Computer Simulation of Flood Works Utilizing Inlet Control and Detention Storage. SWMM Users Group Meeting. November 1978. Annapolis, Maryland. U.S. Environmental Protection Agency. Washington, D. C.

Tottle, C. H., 1979. Stormwater Retention Basins - Research and the Wog Exp. Western Canada Water and Sewage Conference.

Tourbier, J. T. 19__. Water Resources Protection Technology, A Handbook of Measures to Protect Water Resources in Land Development.

Town of Oakville, 1979. Storm Drainage Policies and Criteria. Technical Coordination Committee. Oakville, Ontario.

Tsang, M. T., A. T. Knepp and R. A. Schmaiz, 19__. Evaluation of Flood Risk Factor -Highway Stream Crossing. COE/WRE.

Urban Land Institute, 1976. Lakes and Ponds - Technical Bulletin No. 72. Washington, D. C.

Urban Land Institute, ASCE and National Association of Home Builders, 1975. Residential Stormwater Management - Objectives, Principals and Design Consideration. Washington, D. C.

Urban Land Institute, 19__. Environmental Comment - Stormwater: Liability or Asset? Washington, D.C.

U.S. Army Corps. of Engineers, 1976. STORM: Users Manual. Hydrologic Engineering Center, Davis, California.

U.S. Department of Agriculture, 1977. Minimizing Erosion in Urbanizing Areas. U.S. Department of Agriculture.

U.S. Department of Agriculture, 1975. Urban Hydrology for Small Watersheds. Technical Release No. 55. U.S. Department of Agriculture.

U.S. Department of Agriculture, 1973. HYMO: Users Manual. Agricultural Research Service in Cooperation with Texas A & M. University, College Station, Texas.

U.S. Department of Agriculture, 1972. Soil Conservation Services (SCS) National Engineering Handbook. U.S. Department of Agriculture.

U.S. Department of Transportation, 1980. Underground Disposal of Stormwater Runoff -Design Guidelines Manual. Federal Highway Administration in Cooperation with California Department of Transportation. Washington, D.C.

USEPA, 1977. Urban Runoff Treatment Methods - Volume 1 - Non-Structural Wetland Treatment. Cincinnati, Ohio.

USEPA, 1974. Urban Stormwater Management and Technology: An Assessment. National Environmental Research Agency. Cincinnati, Ohio.

USEPA, 1975. Stormwater Management Model: Users Manual. National Environmental Research Center. Cincinnati, Ohio.

W. L. Wardrop and Associates, 1978. Northwest Sector Drainage Study. A report prepared for the City of Regina. Regina, Saskatchewan.

Waller, D. H., W. A. Coulter, W. M. Carson and D.G. Bishop, 1976. Urban Drainage Model Comparison for a Catchment in Halifax, Nova Scotia. Research Report No. 43. Canada-Ontario Agreement on Great Lakes Water Quality.

Waller, D. H. and W. A. Coulter, 1974. Winter Runoff From an Urban Catchment. Research Report No. 41. Canada-Ontario Agreement on Great Lakes Water Quality.

Wanielista, P. and Y. A. Yousef, 1978. Design and Analysis of Stormwater Retention Basins Using Quality and Quantity Criteria. Proceedings of the International Conference on Urban Storm Drainage. University of Southampton, England.

Wanielista, M. P., 1979. Stormwater Management - Quantity and Quality. Ann Arbor Science Publishers Inc. Ann Arbor, Michigan.

Whipple, W. Jr. (editor), 1974. Urban Runoff - Quantity and Quality. American Society of Civil Engineers.

Whipple, W., N. S. Grigg, T. Grizzard, C. W. Randall, R. P. Shubinski and L. S. Tucker, 1983. Stormwater Management in Urbanizing Areas. Prentice-Hall Inc. Englewood Cliffs, New Jersey.

Williams, J. R. and R.W. Hann, 1973. HYMO Problem-Oriented Computer Language for Hydrologic Modeling - Users Manual. Agricultural Research Service, Department of Agriculture.

Wilson, D., 1983. Capacities of Catch Basin Inlets Based on Use in Alberta. Project for Master of Engineering Degree. University of Alberta. Edmonton, Alberta.

Wisner, P., 1983. IMPSWM Urban Drainage Modelling Procedures. University of Ottawa. Ottawa, Ontario.

Wisner, P., 1980a. Memorandum on S.C.S. Runoff Volume Computations. University of Ottawa. Ottawa, Ontario.

Wisner, P., 1980b. Street Overland Flow and Inlet Control. CSCE.

Wisner, P. 1979. Implementation of SWM Models - Pros and Cons of Standardization. SWMM Users Group Meeting. U. S. Environmental Protection Agency. Washington, D. C.

Wisner, P. and C.E.P'ng, 1982. OTTHYMO-A Model for Master Drainage Plans. University of Ottawa (IMPSWM).

Wisner, P., A. Kassem and P. Cheung, 1979. Review of WRE Transport Models and Presentation of Tests with a New Improved Version. IMPSWM Progress Report No. 1. University of Ottawa.

Wisner, P. and P. Cheung, 1979. Application of the HYMO Model for Runoff Control Studies, Comparison with SWMM, Limitations and Recommendations. IMPSWM Progress Report No. 2. University of Ottawa.

Wisner, P. and A. Kassem, 1979. Comparison of Peak Flows Calculated by the Rational Method and EPA - SWM Model. IMPSWM Progress Report No. 3. University of Ottawa.

Wisner, P. and S. Gupta, 1979. Preliminary Considerations on the Selection of Design Storms. IMPSWM Progress Report No. 4. Ottawa, Ontario.

Wisner, P. and A. Kassem, 1979. Preliminary Comparison of the Simulation of Surcharge by the WRE-IMPSWM Version of SWM TRANSPORT and the Dorsch HVM Model. IMPSWM Progress Report No. 5. University of Ottawa.

Wisner, P. and S. Gupta, 1980. An Assessment of the SCS Procedures for Discharge Computations. IMPSWM Progress Report No. 7. University of Ottawa.

Wisner, P. and P. Cheung, 1980. The Use of SWMM for the Development of Charts for Runoff Control. IMPSWM Progress Report No. 8. University of Ottawa.

Wisner, P. and P. Cheung, 1980. An Assessment of the Rational Method as a Simplified Technique for the Estimation of Runoff Hydrographs. IMPSWM Progress Report No. 10. University of Ottawa.

Wisner, P. and A. Kassem, 1980. Review and Comparison of Routing Methods in Storm Water Modelling. IMPSWM Progress Report No. 13. University of Ottawa.

Wisner, P. and P. Cheung and W. Liang, 1981. Urban Runoff Analysis with Programmable Calculators. IMPSWM Progress Report No. 14. University of Ottawa.

Wisner, P. and C.E.P'ng, 1981. A Linear Reservoir Model for Urban Runoff Using the Programmable Calculater. IMPSWM Progress Report No. 15. University of Ottawa.

Wisner, P. E., et al, 1980. Inlet Control Devices for Storm Sewer Catch Basins. University of Ottawa.

WPCF, 1980. Operation and Maintenance of Wastewater Collection Systems. Manual of Practice No. 7. Washington, D.C.

Wright-McLaughlin Engineers, 1969. Urban Storm Drainage Criteria Manual, Volumes 1 and 2. Urban Drainage and Flood Control District. Denver Regional Council of Governments. Denver, Colorado.

Wullsheleger, Richard E., 1976. Methodology for the Study of Urban Storm Generated Pollution and Control.

Vatagodakumbura, S., A.T.K. Fok and A.C. Rowney, 1981. Some Considerations of Detention Lake Design. Eighth International Symposium of Urban Hydrology, Hydraulics and Sediment Control. University of Kentucky.

Vatagodakumbura, S. et al, 1981. A Review of Methodologies and Techniques of Urban Hydrologic Analysis. A paper for Fifth Canadian Hydrotechnical Conference, Canadian Society of Civil Engineering. Fredericton, New Brunswick.

Vatagodakumbura, S. and A.T.K. Fok, 1979. Application of Stormwater Management -Concepts for a Master Drainage Planning Study. Sixth International Symposium on Urban Storm Runoff. University of Kentucky.

Yen, B.C. and V. T. Chow, 1980. Design Hyetographs for Small Drainage Structures. ASCE Journal of the Hydraulics Division. Vol. 106, No. HY6.

Yen, B.C., 1977. Hydraulics of Storm Sewer Design. University of Kentucky.

Yen, B.C., 1974. Design Storm Sewer Using Rational Method (2 Parts). Water and Sewage Works.

Yen, B.C., P.R. Townsend and W. E. Watt, 1980. Use of Modelling for Plan Stormwater Strategies. Western Canada Water and Sewage Conference Annual Meeting.

APPENDIX A

EXAMPLE OF TYPICAL MUNICIPAL STORMWATER MANAGEMENT DESIGN STANDARDS

STORMWATER MANAGEMENT DESIGN STANDARDS

1.0 PREFACE

1.1 The purpose of these standards is to identify general requirements for stormwater management systems as an aid to municipalities (hereinafter, termed "local authorities") and local authorities developing or upgrading drainage systems.

1.2 The standards noted below have been extracted from servicing manuals and master drainage plans for a number of Canadian municipalities. Consideration of these standards will identify issues which should be addressed in the development of a set of standards for a specific local authority.

1.3 Detailed standards for stormwater management should be developed individually by local authorities based on local considerations. These will vary from community to community depending on the nature of the existing and proposed land uses and the characteristics of local receiving waters.

1.4 These standards should not be considered as rigid requirements where variations will achieve better technical and/or economical solutions. Indeed, engineering consultants and local authorities are encouraged to continuously seek new and better solutions to stormwater management problems.

2.0 GENERAL

2.1 Storm sewers shall be designed as a separate sewer system. These pipes and their appurtenances shall be termed the minor system. The minor system comprises piping, manholes, catch basins and outfall structures.

2.2 The minor system shall convey runoff from snowmelt and rainfall events to an adequate receiving water (stream, lake or pond) without sustaining any surface ponding or excessive surface flows for events up to a 1 in 5 year return period. Where required by the Local Authority, high value commercial areas shall have their minor systems designed to accommodate the 1 in 10 year return period event.

2.3 The major system comprises the street system, detention facilities, parkland and any other land required to convey runoff from events up to a 100 year return period to the receiving water. The major system shall be evaluated in a manner sufficient to determine that no flooding that may cause significant property damage (e.g. flooding of buildings) occurs during the 100 year event.

2.4 Effluent from sanitary sewers and any drainage from industrial, agricultural or commercial operations that may potentially be contaminated shall not be discharged to the storm sewers. Connections for roof leaders shall not be made to the storm sewer system, and connections of weeping tile drainage to the storm sewer should be considered only where soils are permeable and the water table is high.

2.5 Roof drainage from one-family and two-family dwellings shall discharge to grassed or pervious areas. The point of discharge shall be a sufficient distance (e.g. at least 1.0 m) to ensure the water flows away from the building. Roof drainage from apartment buildings, commercial areas, and industrial areas should also be discharged to the surface drainage system except where the building density makes this impractical (e.g. central buisness districts, etc).

2.6 Control shall be provided to minimize sediment discharge to the storm sewers. This shall be in the form of properly graded and surfaced streets and lanes, landscaping, catch basin sumps, sediment control structures at pond and lake inlets, or other means where appropriate.

3.0 DESIGN CRITERIA

3.1 Pipe

3.1.1 Storm sewer pipe shall be designed to convey the design flow when flowing full with the hydraulic gradeline at the pipe crown. All pipe crown elevations shall match (± 0.03 m; ± 1.0 in) at manhole junctions. Where there is limited grade available, and where the proponent has conducted detailed hydraulic analysis, the Local Authority may accept designs based on matching inverts.

3.1.2 Sewer hydraulics shall be calculated using Manning's equation. Manning's n value shall be 0.013 for concrete, vitrified clay and plastic pipes. For corrugated steel, other pipes and open channels the values suggested in "Modern Sewer Design" (AISI, 1980) shall be used but shall not be less than 0.013.

3.1.3 Sewer velocities shall not be less than 0.60 m/s (2.0 f/s) when flowing full. When the flow velocity exceeds 3.0 m/s (10 f/s), special consideration shall be given to minor losses in the system.

3.1.4 All storm sewers shall be of concrete pipe unless approved by the Local Authority. The pipe shall be of sulphate resistant concrete with either a mortar or a rubber ring jointing system. On steep slopes, welded steel pipe or another rigid piping system shall be constructed as directed by the Local Authority.

3.1.5 The minimum inside diameter for storm sewers shall be 300 mm (12 in).

3.1.6 The strength of the pipe shall be sufficient to carry the loads due to trench backfill and due to wheel loads. The strength of pipe shall be calculated on the basis of the external loads, trench conditions and class of bedding provided. Sand bedding is the minimum bedding requirement.

3.1.7 The minimum depth of cover to pipe crown shall be 1.20 m (4 feet).

3.1.8 Changes in flow direction at a manhole shall not exceed 45^{o} in pipes greater than 600 mm (24 in)diameter.

3.1.9 Curved sewers shall match the curvature of the roadway by means of deflection at the joints only. Joint deflections shall be less than the allowable specified for the pipe.

3.2 Manholes

3.2.1 The maximum spacing of manholes shall be 120 m (400 feet). Manholes shall be located at the upstream end of each line, at changes in size or alignment and at all junctions. The downstream invert in a manhole at a change in direction shall be a minimum of 0.03 m (1 in) lower than the lowest upstream invert.

3.2.2 Manholes shall be a minimum of 1200 mm (48 in) in diameter. Precast concrete manholes shall be used. The base shall be constructed of 25 MPa (3,600 psi) sulphate resistant concrete. Manhole frames and covers shall be of cast iron. Grated or standard manhole covers shall be used as required. Galvanized iron safety steps are required.

3.3 Gutters and Catch Basins

3.3.1 Surface water shall not be permitted to run a distance greater than 300 m (1,000 feet) along roadways without provision for interception by the first catch basin. Within the piped drainage system, surface runoff shall not run a distance greater than 120 m (400 feet) without interception.

3.3.2 Surface water shall be intercepted with a number of catch basins such that the inlet capacity is sufficient to receive the design

stormwater flow. Catch basin capacity shall be considered as shown on Table A.1 where values are given for sump conditions and on slope conditions based upon inlet grate type.

3.3.3 Minimum gutter grade shall be at least 0.40%.

3.3.4 All catch basin bodies shall be of either 600 mm (24 in) or 900 mm (36 in) precast concrete sections (sulphate resistant). Where a sump cleaning maintenance program is in effect, the body will be constructed so as to provide a 600 mm sump to trap silt and gravel.

3.3.5 Catch basin leads shall be installed to provide a minimum depth of cover of 1.20 m. All catch basin leads shall discharge directly into storm sewer manholes. The minimum catch basin lead size shall be 250 mm (10 in), with a minimum slope of 1%.

3.3.6 Catch basin frames and covers shall be combination inlet type. Norwood F33, F36 or approved equal shall be used with 600 mm (24 in) concrete sections. Norwood F51, K2 or approved equal shall be used with 900 mm (36 in) concrete sections.

3.4 Detention Facilities

3.4.1 Detention facilities shall be designed as part of both the minor and major drainage systems. They must control the peak runoff conditions for events up to the 100 year return period.

3.4.2 Detention facilities become municipal property. The need for a specific detention facility shall require the approval of the Local Authority. In assessing the need for specific detention facilities, the proponent must consider the impacts of uncontrolled drainage as well as the capital and operating costs of providing control.

3.4.3 Soils investigations specific to the detention facility shall be undertaken to determine appropriate design factors. Where the facility is sited above a shallow aquifer the potential for groundwater contamination must be minimized.

3.4.4 Wet pond detention facilities should be constructed in impervious soils to minimize water losses during dry weather periods. Intruding silt or sand seams should be sealed off.

3.4.5 Where a detention facility is to have multiple functions, its design or should consider the aesthetic implications of shape, grading and landscape features. The Parks and Recreation Departments for the Local Authority should be contacted to provide input to the design from the concept stage on.

3.4.6 An emergency overflow system shall drain to a receiving stream, if possible, for storms greater than the 100 year event.

3.4.7 The effects of the maximum pond water levels shall be considered in the design of the minor system and lot grading. The crown elevations of the pipes in the first manhole upstream of a pond shall be at or above the maximum pond level during the 5 year storm event.

3.4.8 The maximum water level fluctuation for detention ponds in residential areas during the 100 year storm event shall be 2.0 m (6.5 feet). All inhabited building space, including basements, shall be constructed above the 100 year flood level.

3.4.9 In design, wet ponds shall:

a) be located at local low points or adjacent to an existing water course,

b) have a minimum depth of 2.0 m (6.5 feet) at normal water level,

c) have side slopes no steeper than 3 (horizontal) to 1 (vertical) from the bottom of the pond to one meter below normal water level; from here to 5 m (horizontal) beyond the 100 year flood level the side slopes shall be no steeper than 7 (horizontal) to 1 (vertical),

d) have inorganic shoreline treatment for 1.5 m (5 feet) horizontal below and 3.0 m horizontal above the normal water level; the edge treatment shall be compatible with adjacent land use and consider safety, maintenance and access,

e) have inlets and outlets submerged below ice level and above the level of anticipated sediment accumulation; the obverts shall be at least 1.0 m below normal water level,

f) have provision for sediment accumulation at the points of inflow, and for the later removal of the sediment,

g) address all safety issues, and

h) have no dead bay areas.

3.4.10 In design, dry ponds shall:

a) have a low flow bypass for flows from minor events,

b) have a bottom with a minimum longitudinal slope of 0.5%,

c) have a bottom with a minimum lateral slope of 1.0%,

d) have a French drains where longitudinal slopes are less than 0.5% or when lateral slopes are less than 1.0%,

e) have side slopes flatter than 5 (horizontal) to 1 (vertical),

f) have length, width and depth dimensions that are acceptable to the Local Authority's Parks and Recreation Department,

g) have trash bars on inlets and outlets to preclude access by children, and

h) address all safety issues (particularly during operation).

3.4.11 Underground storage tanks shall be considered only if no other economical alternative means of storage is feasible.

3.4.12 Parking lot and rooftop storage shall be considered only for developments where the facilities are part of the project. Proposals which rely on third party implementation shall not be accepted.

3.4.13 Infiltration and evaporation facilities require studies which involve site specific measurements of infiltration, knowledge of the groundwater regime, and an evaluation of the water balance for the system.

3.5 Outfalls

3.5.1 Obverts of outfall pipes shall be at least 150 mm (6 in) above the 5 year flood level in the receiving stream. Inverts of outfall pipes shall be above winter ice level. Otherwise, outfall pipes shall be submerged below the bottom of ice level. In addition, outfalls shall be located to avoid damage from moving ice during breakup.

3.5.2 Drop structures and energy dissipators shall be used where necessary to prevent erosion.

3.5.3 Facilities shall be provided which will prevent entry or access by children.

3.6 Receiving Waters

3.6.1 Measures should be incorporated in new developments to prevent any increase in the amount of downstream erosion.

3.6.2 If a development will cause downstream erosion despite the use of on-site peak runoff rate controls, appropriate measures should be constructed in the downstream areas.

3.6.3 Preservation of watercourse aesthetics and wildlife habitat should be considered in erosion and bank stability work.

3.7 Culverts and Bridges

Culvert and bridge design should consider backwater effects over a range of flows. The design of a hydraulic structure requires assessment of both its nominal design "capacity" and its performance during the 100 year event.

4.0 RAINFALL INTENSITY-DURATION-FREQUENCY CURVES

4.1 The intensity-duration-frequency curves developed by Atmospheric Environment Services of Environment Canada shall be used. The results of their most recent analysis (presently, for the period to 1983) shall be used. Data for these curves are tabulated in Table A.2.

5.0 METHODS OF FLOW ANALYSIS

5.1 Rational Method

5.1.1 The Rational Method of analysis shall be used to determine design flows for piped storm sewer systems of predominantly residential, commercial or industrial land use up to 50 ha (125 acres) in area. Alternatively, computer modelling may be used (see item 5.2).

5.1.2 The formula for the design peak runoff rate shall be:

$$Q = 2.78\ CiA$$

where: Q is the design peak flow rate (L/s)
C is the runoff coefficient
i is the rainfall intensity (mm/hr) corresponding to the time of concentration (see 5.1.5)
A is the area of contributing runoff surface (ha)

5.1.3 The weighted average of pervious and impervious area runoff coefficients shall be estimated from the following equation:

$$C = \frac{C_p A_p + C_i A_i}{A_p + A_i}$$

Where the subscripts p and i indicate the pervious and impervious surfaces, respectively. In these guidelines $C_p = 0.2$ and $C_i = 0.9$.

5.1.4 Where roof leaders discharge to grassed (pervious) areas prior to flowing into the storm sewer system the impervious area may be reduced by the amount of the roof area. The pervious area shall then be correspondingly increased. The minimum C value for residential areas computed in this manner for the 5 year event shall be 0.35. Minimum C values for other land uses and return periods are given in Table A.3.

5.1.5 The duration of rainfall used to determine the intensity is equal to the time of concentration. The time of concentration is comprised of the overland flow time to the storm sewer inlet and the time of travel in the conduit. The overland flow time to curbside in residential and commercial areas shall not exceed 10 minutes in duration (specific overland flow times shall be computed separately for industrial and undeveloped areas). Gutter flow time shall not exceed 5 minutes and shall be estimated based on methods outlined in "Modern Sewer Design" (AISI, 1980). The maximum time of concentration to an upstream inlet for a residential development shall, therefore, be 15 minutes. The time of travel in the conduit shall be based on the pipe-full velocity.

5.2 Computer Modelling

5.2.1 Computer models shall be used to determine design flow conditions in sewer systems with drainage areas larger than 50 ha (125 acres).

They may be used for smaller systems as an alternative to the Rational Method.

5.2.2 Computer models shall be used to determine design flows and the sizing of systems which contain non pipe stormwater management facilities (e.g. detention ponds) or systems that include a significant amount of undeveloped land.

5.2.3 The selection of an appropriate computer model shall be based on an understanding of the principles, assumptions, and limitations in relation to the system being designed. Acceptable computer models are ILLUDAS, USEPA SWMM OTTSWM, HVM, HYMO and OTTHYMO.

5.2.4 Wherever possible, the computer model shall be calibrated. In all analyses, the parameters used, the drainage boundaries, the pipe network and its connectivity shall be clearly identified on an overall drawing, computer printouts and a design summary report.

5.2.5 The design storm hyetograph shall be the 30% distribution for the Prairie Provinces developed by Hogg of Atmospheric Environment Services of Environment Canada (Table A.4). Alternatively, the Chicago method may be used to generate a synthetic design storm.

5.2.6 The duration of event that is critical for systems comprised of pipes only shall be one hour. The duration of the design rainfall event for systems with storage shall be at least 12 hours. Longer durations shall be used, if necessary, to properly assess the post-event drainage of the detention facility.

6.0 MAJOR DRAINAGE SYSTEM ANALYSIS

6.1 The major drainage system shall be assessed with respect to the 1 in 100 year return period event.

6.2 The grading of streets and the layout of the major drainage system shall be assessed, relative to the following guidelines, during the 100 year event:

6.2.1 No building shall be inundated at its ground line.

6.2.2 Continuity of the overland flow routes between adjacent developments shall be maintained.

6.2.3 Arterials should have at least two lanes which are not inundated parallel with the direction of flow. Where the major system crosses an arterial, the depth of flow should be less than 0.05 m.

6.2.4 Collectors should have at least one lane which is not inundated parallel with the direction of flow. Where the major system crosses a collector, the depth of flow should be less than 0.10 m (4 in).

6.2.5 Local roads should have a depth of water no more than 0.05 m above the crown. Where the major system crosses a local road, the depth of flow should be less than 0.15 m (6 in).

6.2.6 The depth of water at curbside should be less than 0.50 m (20 in)for all roadways.

6.2.7 The velocities and depths of flow in the major drainage system shall not exceed the values outlined in Table A.5.

6.3 The grading of lots shall meet the following requirements:

6.3.1 The minimum slope in the back yard shall be 2%.

6.3.2 If the back yard area slopes toward the house, provision must be made to keep the runoff at least 3 m (10 feet) from the house and direct it to the street.

6.3.3 Reverse driveways shall not be permitted unless the runoff can be directed away from the structure and off the property in a controlled fashion.

TABLE A.1
CATCH BASIN CAPACITIES (L/s)

Catch Basin	Flow Condition	
	Continuous Slope	Sump Condition
F33	11	20
F36	10	23
F38, F39	20	38
F51	40	80
K2	25	45

Notes:

1. Capacities for F33 and F36 are based on hydraulic studies conducted by the City of Edmonton (Townsend and Moss, 1980). Sump capacity is based on 5 cm (2 in) head. Continuous slope capacity based on gutter flow of 40 L/s (1.4 cfs).
2. Capacities for others are based on the work of Wilson (1983). Sump capacity based on 65% of theoretical capacity for 5 cm (2 in) head. Continuous slope capacity based on ratios for F33 and F36 tests.
3. Capacities can be considered to be double those shown in the table when assessing the 100 year event.

TABLE A.2

INTENSITY DURATION FREQUENCY DATA

_____, ALBERTA

Duration (minutes	Rainfall Intensity (mm/hr)		
	5 Year	10 Year	100 Year
10	78	95	147
11	75	92	142
12	72	88	137
13	70	85	131
14	67	81	126
15	64	78	121
16	62	76	118
17	61	75	116
18	60	73	113
19	58	71	111
20	57	70	108
21	55	68	106
22	54	66	103
23	52	65	101
24	51	63	98
25	50	61	96
26	48	60	93
27	47	58	91
28	45	56	88
29	44	55	86
30	42.4	53	83
60	25.1	30.8	48.5
120	15.0	18.1	27.7
360	7.37	8.8	13.6
720	4.36	5.26	8.08
1440	2.64	3.17	4.85

Notes:

1. Based on AES data for period ___ to 198_ (__ years).
2. Values for 11 to 14 minutes and 16 to 29 minutes are interpolated.

TABLE A.3
RECOMMENDED RUNOFF COEFFICIENTS

Land Use or Surface Characteristics	Rainfall Event Return Period			
	2 Year	5 Year	10 Year	100 Year
Residential	0.30	0.35	0.40	0.60
Apartments	0.65	0.70	0.70	0.80
Downtown Commercial	0.85	0.85	0.90	0.90
Neighbourhood Commercial	0.60	0.65	0.70	0.80
Lawns, Parks, Playgrounds	0.10	0.20	0.25	0.30
Undeveloped Land (Farmland)	0.05	0.10	0.15	0.20
Paved Streets	0.90	0.90	0.95	0.95
Gravel Streets	0.15	0.25	0.35	0.65

Notes:

1. Values are recommended minimum for use in the Rational Method.
2. Where specific land surfaces are known for Apartment and Commercial areas, the runoff coefficient may be determined based on the surface characteristics for the ultimate development conditions.
3. Runoff coefficients for industrial land use must be estimated based on ultimate land use condition.

TABLE A.4
DESIGN STORM HYETOGRAPHS

1 Hour Design Storm			12 Hour Design Storm		
Period Ending (minutes)	Cumulative Rain (%)	Discrete Rainfall (%)	Period Ending (hours)	Cumulative Rain (%)	Discrete Rainfall (%)
0	0	0	0	0	0
5	12	12	1	15	15
10	32	20	2	34	19
15	54	22	3	51	17
20	74	20	4	66	15
25	85	11	5	83	17
30	90	5	6	90	7
35	95	5	7	96	6
40	98	3	8	98	2
45	99	1	9	100	2
50	100	1	10	100	0
55	100	0	11	100	0
60	100	0	12	100	0

Notes:

1. Data from Atmospheric Environment Services of Environment Canada.
2. Design storms are the 30% distributions for the Prairie Provinces.

TABLE A.5
PERMISSIBLE DEPTHS FOR SUBMERGED OBJECTS,

Water Velocity (m/s)	Permissible Depth (m)
0.5	0.80
1.0	0.32
2.0	0.21
3.0	0.09

Note: Based on a 20 kg (45 pound) child and concrete lined channel. Larger persons may be able to withstand deeper flows.